Reinforcement for Modern Concrete Structures

Khan Mahmud Amanat

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There are a lot of excellent textbooks on the design of reinforced concrete structures. However, based on professional experience of more than thirty years, the author feels that a separate book dedicated only to discuss the properties, behavior and use of reinforcement in concrete construction is highly justified. Conventional textbooks on concrete structures focus primarily on the mechanics of reinforced concrete design. Properties of reinforcement are discussed in a limited manner – only those which are deemed just adequate for discussing the theory of concrete mechanics. Typically, such books contain little or no background information or explanation as to why the various code provisions or rules regarding the reinforcement are imposed or formulated. It is observed that the application of modern materials and technologies in reinforcement manufacturing does not get proper attention or consideration in the engineering practices of many professionals. Based on the interaction with many engineers engaged in the design and construction of concrete structures, the author feels that there is a lack of the essential background knowledge of the engineering properties and behavior of concrete reinforcement among many of them. And such a lack of knowledge often leads to improper implementation of the code design provisions. Limited scope of our existing engineering curricula is primarily responsible for this. This book has been written with an aim to fill this gap and to make engineering students and practicing engineers more up-to-date.

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About the Author

Dr Khan Mahmud Amanat is Professor in the Department of Civil Engineering, Bangladesh University of Engineering & Technology (BUET). In his more than a 30-year long career as an academician, Dr Amanat has published several research articles on concrete and steel in many international journals, some of which also received best paper awards. He supervised several master's and PhD researches. Dr Amanat is very active in developing and modernizing the civil engineering curricula of BUET. His current research interests include the behavior of steel slit dampers and shear links under cyclic loading, optimized design of post-tensioned PC-girders, controlled rocking self-centering steel braced frames, soft-storied buildings, repair and retrofitting structures using CFRP laminates, behavior of bolted flanged connection subjected to bending, replaceable steel coupling beams, recycled brick concrete aggregate, estimation of in-situ strength of brick concrete columns, the behavior of engineered cementitious composite made of locally available materials, evaluation of the strength of fiber reinforced concrete by core testing, etc. Dr Amanat is also contributing to the development of the nation through consultancy and advisory services. He had been actively involved in updating and developing the Bangladesh National Building Code 2020. He acted as a consultant or expert in a number of important national mega projects like the Padma Bridge Project, the repair of Bangabandhu Jamuna Bridge, Meghna Bridge, the design of bridges and flyovers, etc. As an expert academician, he has served as the Chairman of the Civil Engineering Division of the Bangladesh Standards and Testing Institute (BSTI) as well as a member of various committees formed by the government. As an experienced academician and professional, he also conducted several seminars and workshops at home and abroad.

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Preface

There are a lot of excellent textbooks on the design of reinforced concrete structures. However, based on his experience teaching and designing structures for more than 30 years, the author feels that a separate book dedicated only to discussing the properties, behavior and use of reinforcement in concrete construction is highly justified. Conventional textbooks on concrete structures focus primarily on the mechanics of reinforced concrete design, where properties of reinforcement are discussed in a limited manner. Such books generally cover only the topics which are deemed to be just adequate for discussing the theory of concrete mechanics. Typically, these books contain little or no background information or explanation as to why the various code provisions or rules regarding the reinforcement are imposed or formulated. Based on interactions with many engineers engaged in the design and construction of concrete structures, the author feels that there is a lack of essential background knowledge of the engineering properties and behavior of concrete reinforcement. He has also observed that the application of modern materials and technologies in reinforcement manufacturing does not get proper attention or consideration in engineering practices. And such a lack of knowledge often leads to improper implementation of the code design provisions. Sometimes this may result in an unsafe structure too. The engineers or designers are not to be blamed for this. Instead, it is the limitation of our existing engineering curricula that is primarily responsible. This book has been written with an aim to fill this gap and to make engineering students and practicing engineers more up-to-date.

Concrete comprises six ingredients – coarse aggregate, fine aggregate, cement, water, admixture and reinforcement. Of these six ingredients, only the steel reinforcement is responsible for making a concrete structure seismically resilient – the role of other ingredients is much less significant. Seismic-resistant concrete structures can only be achieved by increasing the rebar material's ductility and prescribing proper detailing in laying out or fabricating the rebar, which will ensure structural ductile behavior. Thus, the importance of understanding the behavior of rebar cannot be overstated. Various aspects of reinforcement such as some historical aspects, engineering properties, testing methods and procedures, characteristics of high strength rebar, code application, placing and fabrication details, usage of coating on rebar for corrosion protection and rebar splicing and coupling, alternative rebar materials, etc. are discussed in ten different chapters in this book.

After a brief historical account of using steel bars as reinforcement in concrete structures in Chapter 1, the physical, mechanical and chemical properties of steel reinforcement are discussed in Chapter 2. Detailed discussions on physical and mechanical properties, chemical compositions, etc. are made so that the reader shall have a very clear idea about the engineering properties of rebar and their significance in the design of reinforced concrete structures. Laboratory procedures for testing rebar to determine its various engineering properties are discussed in Chapter 3.

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In the author's opinion, these two chapters contain some essential information that every engineering student and professional should be aware of.

A brief review of the application of high-strength rebar around the world is made in Chapter 4. Though steel grades higher than Grade 60 (60 ksi yield strength) were introduced in the 1960s, seismic use of rebar was restricted to maximum Grade 60, which continued until 2019. However, the use of higher-grade steel for non-seismic applications has been allowed for the last few years. In the 2019 version of the ACI 318 code, the use of Grade 80 rebar (550 MPa) in seismic applications was allowed for the first time in the United States. This marked a major enhancement to seismic code provisions. To facilitate the engineers' and students' understanding of these recent enhancements, detailed but focused discussions are made in Chapter 5.

Details about the placing and fabrication of rebar are discussed in Chapter 6 based on the recent ACI provisions and recommendations of CRSI. Based on the author's experience, a few typical reinforcement details for common reinforced concrete construction are also suggested in this chapter. Though rebar splicing is a part of rebar detailing, considering its importance in construction, a separate chapter, Chapter 7, is dedicated to this topic. It is expected that practicing engineers shall be able to develop a good understanding of the use of rebar coupler or welding as an alternative to lap splicing based on the information presented in this chapter.

Owing to high initial cost, the use of stainless steel rebar for concrete structures is generally discouraged. But its excellent corrosion resistance makes it a good contender as an alternative to common black MS rebar, especially in situations where corrosion resistance and durability are important. It is expected that the reader shall have a good introduction to stainless steel rebar from Chapter 8.

Chapter 9 is dedicated to the discussion of coated reinforcing steel such as zinc or epoxy coated rebar which provides a balance between cost and durability of concrete. Brief coverage of other types of coatings is also made. Characteristics, performance and other engineering properties of such rebar are discussed in this chapter.

Chapter 10 is the last chapter of this book and is dedicated to the discussion of non-metallic reinforcement for concrete. The use of such rebar is gradually gaining popularity owing to some of its superior engineering characteristics, including high tensile strength and very good corrosion resistance. Various kinds of non-metallic rebar made from glass, carbon and basalt fiber etc. are discussed in this chapter.

The encouragement and initiative to write this book came from Engr. M. Firoze, former Head of Marketing and Product Development of BSRM. Special thanks also go to BSRM authority for supporting this work. The author would like to thank his past and present students and colleagues for their constructive comments, criticisms and for their feedback and encouragement in writing this book.

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List of Abbreviations

AASHTO	American Association of State Highway and Transportation Officials
ACI	American Concrete Institute
AFRP	Aramid fiber reinforced polymer
AISC	American Institute of Steel Construction
AISI	American Iron and Steel Institute
ANSI	American National Standards Institute
ASCE	American Society of Civil Engineers
ASD	Allowable strength design
ASTM	American Society for Testing and Materials
ATC	Advanced Technology Council
AWS	American Welding Society
BDS	Bangladesh Standard
BNBC	Bangladesh National Building Code 2020
BS	British Standard
BSRM	Bangladesh Steel Rerolling Mills (Ltd.)
BSTI	Bangladesh Standards and Testing Institute
CE	Carbon equivalent
CFM	Continuous filament mat
CFRP	Carbon fiber reinforced polymer
CHZ	Calcium hydroxyzincate
CJP	Complete joint penetration
CRE	Chemically reactive enamel
CRP	Same as CFRP
CRSI	Concrete Reinforcing Steel Institute
DIN	Deutsches Institut für Normung
DOT	Department of Transportation
ECR	Epoxy coated rebar
FBE	Fusion bonded epoxy
FCAW	Flux cored arc welding
FDOT	Federal Department of Transportation
FHWA	Federal Highway Administration
FIB	Fédération Internationale du Béton
FRP	Fiber reinforced polymer
GFRP	Glass fiber reinforced polymer
GMAW	Gas metal arc welding
GTAW	Gas tungsten arc welding
GWP	Global warming potential
HDG	Hot dipped galvanizing
HDPE	High density polyethylene
HPC	High performance concrete
HSRB	High strength reinforcing bars

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HSS	High strength steel
HZMB	Hong Kong-Zhuhai-Macau Bridge
ICC	International Code Council
ISO	International Standards Organization
JIS	Japan Industrial Standard
LCA	Life cycle assessment
LCC	Life cycle cost
LRFD	Load and resistance factor design
LVDT	Linearly variable displacement transducer
NEHRP	National Earthquake Hazards Reduction Program
NIST	National Institute of Standards and Technology
NSM	Near surface mounting
NZS	New Zealand Standards
PC	Prestressed concrete
PCA	Portland Cement Association
PCI	Prestressed Concrete Institute
PREN	Pitting resistance equivalent number
PVC	Polyvinyl chloride
RC	Reinforced concrete
SDC	Seismic design category
SEAOC	Structural Engineers Association of California
SFRS	Seismic force resisting system
SMAW	Shielded metal arc welding
SSR	Stainless steel rebar
TMT	Thermo-mechanical treatment
TRB	Transportation Research Bulletin
TRRL	Transport and Road Research Laboratory
UK	United Kingdom
UNS	Unified numbering system
US	United States
USD	Ultimate strength design
UTM	Universal testing machine
WPS	Welding procedure specification
psi	pounds (force) per square inch
ksi	kilo pounds (kips) per square inch

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Symbols and Notations

A	cross sectional area of rebar.
A_g	gross column cross sectional area.
A_{gt}	percentage elongation at maximum force.
A_{hs}	total cross-sectional area of hooked or headed bars being developed.
$A_{s,min}$	minimum area of flexural reinforcement.
A_{st}	cross sectional area of longitudinal reinforcement.
A_{th}	total cross-sectional area of ties or stirrups confining hooked bars.
A_{tr}	total cross-sectional area of all transverse reinforcement that is within the spacing s and that crosses the potential plane of splitting through the reinforcement being developed.
A_{we}	effective area of weld.
a	rib/deformation height.
α_f	the ratio of the flexural rigidity of a beam section to the flexural rigidity of a width of slab bounded laterally by centerlines of adjacent panels, if any, on each side the beam ($\alpha_f = E_{cb}I_b / E_{cs}I_s$).
β	rib angle.
$\beta_{1 \text{ or } 2 \text{ or } 3}$	rib angle.
b_w	web width beam or diameter of circular section.
c	deformation spacing.
c_1	dimension of rectangular or equivalent rectangular column, capital or bracket measured in the direction of the span for which moments are being determined.
c_2	dimension of rectangular or equivalent rectangular column, capital, or bracket measured in the direction perpendicular to c_1.
c_b	lesser of: (a) the distance from center of a bar or wire to nearest concrete surface, and (b) one-half the center-to-center spacing of bars or wires developed.
d	effective depth defined as distance from extreme compression fiber to centroid of longitudinal tension reinforcement.
d_b	nominal rebar diameter.
δ	rebend angle.
E_{cb}	Young's modulus of beam concrete.
E_{cs}	Young's modulus of slab concrete.
E_s	Young's modulus of steel.
ε_a	proportional limit strain.
ε_{emf}	elongation (strain) at maximum force.
ε_t	net tensile strain in extreme layer of longitudinal tension reinforcement at nominal strength.
ε_{ty}	value of net tensile strain in the extreme layer of longitudinal tension reinforcement used to define a compression-controlled section.
ε_u	ultimate strain or fracture strain.

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ε_y	yield strain.
F_{EXX}	tensile strength of weld material.
F_{nw}	nominal strength of weld material.
F_{up}	upper force for fatigue testing.
F_r	force range for fatigue testing.
f	frequency of fatigue test loading.
f_a	proportional limit stress.
f'_c	design compressive strength of concrete.
f_r	modulus of rupture of concrete.
f_u	ultimate tensile strength of reinforcement.
f_y	specified yield strength of non-prestressed reinforcement.
f_{yt}	specified yield strength of transverse reinforcement.
φ	strength reduction factor.
g	chord gap.
γ	bend angle.
h	height of rebar deformation, overall thickness, height, or depth of member.
h_x	maximum center-to-center spacing of longitudinal bars laterally supported by corners of crossties or hoop legs around the perimeter of a column or wall boundary element.
I_b	gross moment of inertia of beam cross section about centroidal axis.
I_s	gross moment of inertia of slab section about centroidal axis.
K_{tr}	transverse reinforcement index [$=40A_{tr}/(sn)$].
L	length.
l_d	development length in tension of deformed bar.
l_{dh}	development length in tension of deformed bar or deformed wire with a standard hook, measured from outside end of hook, point of tangency, toward critical section.
l_{dt}	development length in tension of headed deformed bar, measured from the bearing face of the head toward the critical section.
l_n	longest clear span in the floor system.
l_o	length, measured from joint face along axis of member, over which special transverse reinforcement must be provided.
l_{sc}	compression lap splice length.
l_{st}	tension lap splice length.
l_w	length of entire wall, or length of wall segment or wall pier considered in the direction of shear force.
λ	modification factor to reflect the reduced mechanical properties of lightweight concrete relative to normal weight concrete of the same compressive strength.
M_n	nominal flexural strength at a section.
m	mass.
n	number of bars or wires being developed along the plane of splitting.
P_o	nominal axial compressive strength of column at zero eccentricity.

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P_u	factored axial force; to be taken as positive for compression and negative for tension.
R_a	required strength using nominal load combinations.
R_e	yield strength.
$R_{ek.L}$	the lower characteristic value of the yield stress determined from a series of tensile tests.
$R_{ek.U}$	the upper characteristic value of the yield stress determined from a series of tensile tests.
R_m	value of the maximum tensile strength determined from a single tensile test.
R_n	nominal strength.
R_u	required strength using factored load combinations.
r	radius of curvature at the root of surface deformation of rebar.
ρ	density, steel ratio.
s	maximum spacing on centers of transverse reinforcement within l_d, center-to-center spacing of items, such as longitudinal reinforcement, transverse reinforcement, tendons or anchors.
s_o	center-to-center spacing of transverse reinforcement within the length l_o.
V_u	factored shear force at section.
Ω	safety factor.
ψ_c	factor used to modify development length based on concrete strength.
ψ_e	factor used to modify development length based on reinforcement coating.
ψ_s	factor used to modify development length based on grade of reinforcement.
ψ_b	factor used to modify development length of hooked and headed bars based on side cover and confinement.
ψ_p	factor used to modify development length for headed reinforcement based on parallel tie reinforcement.
ψ_f	factor used to modify development length based on confining reinforcement.
ψ_s	factor used to modify development length based on reinforcement size (dia.).
ψ_t	factor used to modify development length for casting location in tension.

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1 Introduction

1.1 GENERAL

The quest for using high-strength or high-performance materials for building construction began at the beginning of modern civilization, and will continue into future too. In Bangladesh, as well as in other countries of the world, reinforced concrete is one of the major forms of building and other constructions. In the area of tall building construction, steel framing had been the primary choice of construction in developed countries. However, the 9/11 incident changed the scenario significantly. Now, reinforced concrete is regarded as one of the most reliable and versatile materials for the construction of high-rise structures. The 132 storied 632-meter high Shanghai Tower (Figure 1.1) is standing upon the strength of its reinforced concrete core. The construction of such buildings would not have been possible without the use of high-strength materials.

FIGURE 1.1 Shanghai Tower.

In the reinforced concrete industry, there is a growing interest in using reinforcing steel with greater strength for specific applications. This interest is primarily motivated by the desire to alleviate congestion, particularly in structures with a high seismic design category. There are additional applications where a high-strength bar can improve construction efficiency or, when combined with high-strength concrete, permit reinforced concrete to be used in more demanding applications. In 2004, the adoption of the ASTM A1035 standard established the material specification that permits 100 ksi or even 120 ksi reinforcing steel. The most recent version of ACI 318 allows the use of such high-strength bars on some occasions. Steel grades 60 and 75 are currently utilized in the vast majority of concrete design and construction. For concrete structures, ACI 318 specifications are followed in Bangladesh. Provisions of the 2008 version of ACI 318 are adopted in BNBC 2020, which allows both ASTM A615 and

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ASTM A706 reinforcing materials. ASTM A706 specifies two grades – Grade 60 and Grade 80 with tighter yield and ductility requirements than similar A615 rebar. While Grade 60 rebar can be universally used in all concrete applications, ACI 318 used to place some restrictions on the use of Grade 75 or Grade 80 rebar, specifically in seismic applications until the 2014 version of the code. In the 2019 version of ACI 318, Grade 80 rebar is allowed to be used in the seismic applications of concrete frames, provided that certain detailing conditions are met. However, the Bangladesh National Building Code 2020, formalized in February 2021, still does not permit the use of Grade 80 rebar in special or high seismic applications. In this book, a detailed discussion on the use of these high-strength rebar in various concrete applications is made.

1.2 HISTORY OF REINFORCED CONCRETE

The history of using concrete in construction may be as old as human civilization itself. However, it was not until recent centuries that man began to examine the use of reinforcing elements in concrete. The process of reinforcing concrete consists of embedding steel reinforcing bars, also known as rebar, in concrete before it sets in order to support the concrete and resist tensile stress. In modern concrete construction, rebar is also used in concrete to provide confinement to concrete, especially in beams and columns, to make it perform better under seismic actions.

Joseph-Louis Lambot (Joseph-Louis Lambot, Wikipedia 2021) of France may be regarded as the inventor of reinforced concrete who, in 1848, used iron reinforcement with cement mortar to construct water tanks and troughs. A few years later, in 1954, W.B. Wilkinson (Brown 1966) in the United Kingdom used iron bars and wire rope to strengthen the concrete floors of a two-story building he was building for his employees. This building is generally regarded as the first building to use reinforced concrete for floors. In the same era, French entrepreneur and industrialist François Coignet made widespread use of reinforced concrete in the construction of buildings (François Coignet, Wikipedia 2021). American inventor George Bartholomew is credited with the invention of concrete pavement in 1891 (George Bartholomew, Wikipedia 2022).

The first materials specifications for steel reinforcing bars were developed in 1910 (Lane and Kleinhans 2016). New *lugged* or deformed bars were introduced in the 1910s–1930s period. Standard specification ASTM A15 included rebar Grades 33 and 50 in the year 1911. Later in 1914, Grade 40 was included in it. AASHTO, in 1924, issued its first standard for reinforcing steel AASHTO M31. A diamond pattern bar that was first used in the 1920s still meets current specifications and is in use today. Reinforcing bar producers and fabricators have their own association, the Concrete Reinforcing Steel Institute, abbreviated as CRSI, which was established in 1924. In 1947, ASTM A305 was published, which included several specifications for rebar deformation patterns. ASTM A432 was published in 1959, which included Grade 60 rebar.

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1.3 HISTORY OF HIGH-STRENGTH REBAR

The design of reinforced concrete structures in the United States and around the world was dominated for many years by the use of steel reinforcement with yield strength, f_y, equal to 40 ksi (275 MPa) and, since 1970, 60 ksi (ATC 115 2014). American Concrete Institute (ACI) Committee 318 has a history of revising provisions to include the use of higher strength reinforcement than was readily available in the past. As of the 1950s and early 1960s, Intermediate Grade (Grade 40) reinforcement and Hard Grade (Grade 50) reinforcement had been available for about 50 years and was commonly used. In 1959, the American Society for Testing and Materials (ASTM) specifications A432 and A431 were published, which introduced Grade 60 and Grade 75 reinforcement, respectively. The 1963 version of ACI 318 allowed the use of steel bars with a yield strength of 60 ksi. ASTM A615, *Standard Specification for Deformed and Plain Carbon-Steel Bars for Concrete Reinforcement*, which included Grades 40, 60 and 75, was introduced in 1968. In the late 1950s and 1960s, the Portland Cement Association (PCA) conducted a series of tests, reported in eight parts, that examined beams, girders and columns. These tests covered flexural strength, control of flexural cracking, compression splices in columns and fatigue (Hognestad 1961, 1962; Gaston and Hognestad 1962). Reinforcement strengths ranged from 55 ksi to 120 ksi. These early tests were considered in changes to the 1971 edition of ACI 318 when the upper limit for yield strength was increased to 80 ksi, even though there were no ASTM specifications for reinforcement with yield strengths more than 75 ksi at that time (ATC 115).

In the 1971 edition of ACI 318, the maximum specified yield strength was restricted to 60 ksi for reinforcement in special seismic systems, and this limit was in effect until the release of ACI 318-19 which is reapproved in 2022. The Structural Engineers Association of California (SEAOC) developed a specification for reinforcement with more restrictive tensile properties and chemistry controls, published as ASTM A706, *Standard Specification for Low-Alloy Steel Deformed and Plain Bars for Concrete Reinforcement*. ASTM A706 was permitted in the 1977 edition of ACI 318. It became required for reinforcement in special seismic systems in 1983, but an exception was made for the use of A615 reinforcement if certain mechanical properties were met.

The 2009 versions of ASTM A615 and A706 specifications were the first to include requirements for Grade 80 reinforcement. ACI Committee 318 adopted these specifications without restriction in the main body of ACI 318-11 because the use of Grade 80 reinforcement was already permitted, except in special seismic applications. The trend continued until the next version of the specification, i.e. ACI 318-14 2014). However, until the release of the 2019 version of ACI 318 (ACI 318-19), Grade 80 reinforcement was not permitted for use in special moment-resisting frames and special structural walls due to the perceived lack of test data for cyclically loaded members with Grade 80 reinforcement. In the most recent version of the code, i.e. ACI 318-19, the use of Grade 80 rebar in special seismic applications is permitted.

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QUESTIONS

1-1. What are the reasons behind growing interest in using high-strength reinforcing steel?

1-2. Discuss the gradual adoption of high-strength rebar in building codes.

1-3. Give a brief account of the history of reinforced concrete.

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2 Properties of Mild Steel Rebar

2.1 PHYSICAL PROPERTIES

For reinforced concrete structures, mild steel reinforcement, often termed rebar, comes as round bars with surface deformations. The use of rebar with a plain surface is now obsolete and is virtually non-existent. Mild steel rebar is a hot-rolled product generally produced from billets. Immediately after hot rolling, it is often subject to thermo-mechanical treatment (TMT) in the rolling mill to produce rebar with higher mechanical strength.

2.1.1 Rebar Size

For the convenience of design and construction, rebar comes in many *sizes* where the *size* generally refers to its average or nominal diameter. The size or nominal diameter of the rebar is standardized in international codes, e.g., in ASTM (United States) or ISO (Europe) standards.

In the United States, rebar size is traditionally designated as numbers, e.g., #3, #6 etc., where the numerical value indicates multiples of one-eighth of an inch. That means a #5 bar has a nominal diameter of ⅝ inch = 0.625 inch. In later editions of ASTM, bar sizes are also designated in metric notation by their average diameter in millimeters rounded to a nearest whole number. For example, a #4 bar has 0.5 inch or 12.7 mm diameter which is given a metric designation of 13. Similarly, #5 has an average diameter of 15.875 mm, which is designated as 16. The standard American bar sizes are defined in ASTM A615/A615M as well as in other ASTM standards. In order to match the European sizes, there is also a set of alternate bar sizes defined in the ASTM standards. The ASTM sizes are shown in Table A.1 in the Appendix. Size requirements of ASTM A706 or A1035 standards are the same.

In European ISO standards, rebar sizes directly indicate their average or nominal diameter in millimeters. The standard ISO bar diameters are defined in ISO 6935-2:2015 standard. The ISO standard is also adopted in Bangladesh as the national standard, which is designated as BDS ISO 6935-2:2016. The ISO bar sizes are shown in Table A.2 in the Appendix.

Since the rebar surface has deformations, size or diameter generally refers to a nominal or average measurement. In these standards, the nominal diameter is defined as the diameter of a perfectly round bar which has the same mass per unit length as that of a deformed bar.

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2.1.2 Weight

The weight of rebar generally refers to its weight per unit length which is expressed as lb/ft or kg/m. It is an important parameter which is used in the quantity analysis of a concrete structure. In the presence of surface deformations, direct measurement of rebar diameter is difficult. In such a situation, the weight (per unit length) of the rebar can be conveniently used to calculate the average diameter considering that the steel has a density of 7850 kg/m^3 or 490 lb/ft^3. If L is the length of a rebar sample, m is its mass, and ρ is the density, then the average diameter, d_b, of rebar can easily be calculated using the following Equation 2.1.

$$d_b = \sqrt{\frac{4m}{\pi \rho L}} \tag{2.1}$$

The procedure is simple. Take a piece of rebar and measure its length and mass. Then use Equation 2.1 to determine the average diameter. Standardized unit weights corresponding to the standard bar designations or diameters are given in Tables A.1 and A.2 in the Appendix. In ASTM, it is considered acceptable if the measured unit weight of sample rebar does not fall below 94% of the values shown in Table A.1. According to these limits, the permissible weight and size of various rebar designations can be calculated. A summarized list is shown in Table 2.1. It may be mentioned that ASTM standards only restrict the minimum weight and there is no upper limit imposed on weight.

The limits of acceptability of unit weight in ISO standard are shown in Table A.2. According to these limits, the permissible weight and size of various rebar can be calculated. A summarized list is shown in Table 2.2.

TABLE 2.1
Minimum permissible weight and diameter of ASTM A615/A615M and A706/A706M rebar

Bar designation	Minimum permissible weight [mass], lb/ft [kg/m]	Minimum permissible average diameter, in. [mm]	Alternate bar designation	Minimum permissible weight [mass], lb/ft [kg/m]	Minimum permissible average diameter, in. [mm]
3 [10]	0.353 [0.525]	0.363 [9.22]	10	0.389 [0.579]	0.382 [9.70]
4 [13]	0.623 [0.927]	0.483 [12.27]	12	0.561 [0.835]	0.458 [11.63]
5 [16]	0.980 [1.458]	0.606 [15.39]	16	0.997 [1.484]	0.611 [15.52]
6 [19]	1.412 [2.101]	0.727 [18.47]	20	1.558 [2.319]	0.764 [19.41]
7 [22]	1.921 [2.859]	0.848 [21.54]	25	2.434 [3.622]	0.954 [24.23]
8 [25]	2.510 [3.735]	0.969 [24.61]	28	3.053 [4.543]	1.069 [27.15]
9 [29]	3.196 [4.756]	1.094 [27.79]	32	3.987 [5.933]	1.221 [31.01]
10 [32]	4.045 [6.020]	1.230 [31.24]	36	5.046 [7.509]	1.374 [34.90]
11 [36]	4.994 [7.432]	1.367 [34.72]	40	6.230 [9.272]	1.527 [38.79]
14 [43]	7.191 [10.702]	1.640 [41.66]	50	9.735 [14.488]	1.909 [48.49]
18 [57]	12.78 [19.019]	2.187 [55.55]	60	14.018 [20.862]	2.290 [58.17]

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TABLE 2.2
Maximum and minimum permissible weight and diameter of ISO 6935-2:2015 rebar

Nominal diameter, mm	Maximum permissible weight, kg/m	Minimum permissible weight, kg/m	Maximum permissible average diameter, mm	Minimum permissible average diameter, mm
6	0.240	0.204	6.24	5.75
8	0.426	0.363	8.31	7.67
10	0.654	0.580	10.30	9.70
12	0.942	0.835	12.36	11.63
14	1.269	1.149	14.34	13.65
16	1.658	1.500	16.39	15.59
20	2.591	2.344	20.49	19.49
25	4.010	3.701	25.50	24.49
32	6.569	6.064	32.63	31.35
40	10.264	9.475	40.79	39.19
50	16.038	14.804	50.99	48.99

2.1.3 Surface Deformations

Surface deformations are protrusions on the rebar surface placed in a regular pattern. The function of deformation is to create a strong mechanical bond between concrete and rebar so that no slip occurs. This is an essential feature of steel reinforcement used for concrete. In concrete mechanics, it is a general assumption that the steel remains fully bonded with concrete and that the strain in concrete is the same as that in steel rebar. Deformations play an important role in holding this assumption true and functional. Examples of rebar with surface deformations and ribs are shown in Figure 2.1.

There are endless geometric designs of deformations used by rebar manufacturers. In order for deformations to perform their intended functions, they must meet some requirements. These requirements are outlined in standards, e.g., ASTM or ISO provisions.

Requirements of deformation in ASTM A615, A706 and A1035 standards are as follows:

a. Deformations must be spaced at substantially uniform distances along the bar (c in Figure 2.2). The size, shape and pattern of the deformations on opposite sides of the bar must be similar.
b. The deformations must be positioned in relation to the bar's axis so that the included angle (β_1, β_2 or β_3 in Figure 2.2) is greater than 45°. When the line of deformations forms an included angle with the bar's axis between 45° and 70°, the deformations must alternately reverse in direction on each side, or those on one side must reverse in the direction from those on the opposite

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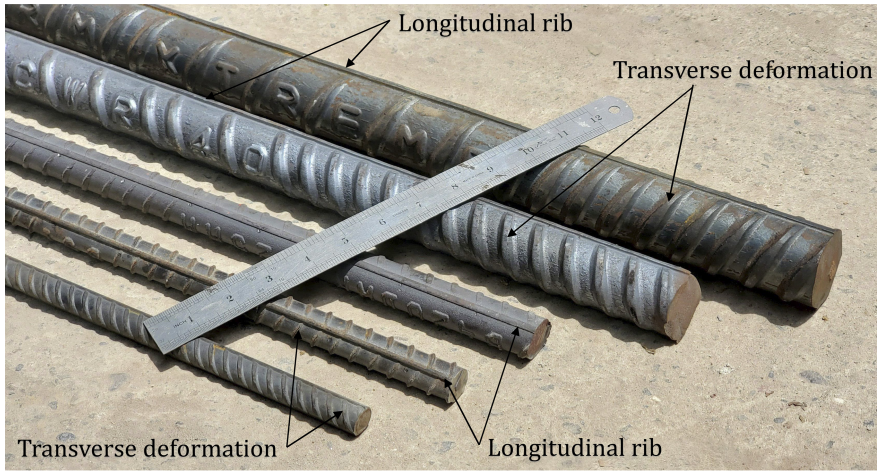

FIGURE 2.1 Rebar with surface deformations and ribs.

side. A reversal in direction is not required when the line of deformations is over 70°.

c. The average distance or spacing (c) between deformations on each side of the bar shall not exceed seven-tenths of the bar's nominal diameter.
d. The total length of deformations must be such that the distance (measured as a chord, g in Figure 2.2) between the ends of deformations does not exceed 12.5% of the nominal perimeter of the bar. Where ends terminate in a rib, the width of the rib shall be regarded as the distance between these ends. The sum of the gaps shall not exceed 25% of the nominal circumference of the bar. The nominal perimeter of the bar shall be considered as 3.1416 times its nominal diameter.
e. The spacing, height and gap of deformations shall comply with the specifications outlined in Table A.3 in the Appendix.

Requirements of deformations or ribs in ISO 6935-2:2015 are as follows:

a. Bars with ribs must have transverse ribs. The presence of a longitudinal rib is not mandatory.
b. At least two rows of transverse ribs shall be evenly spaced around the perimeter of the rebar cross-section. Even distribution of the transverse ribs row along the entire length of the bar is required, with the exception of the marking area.
c. Ribs shall meet the requirements in Table A.4 in the Appendix.

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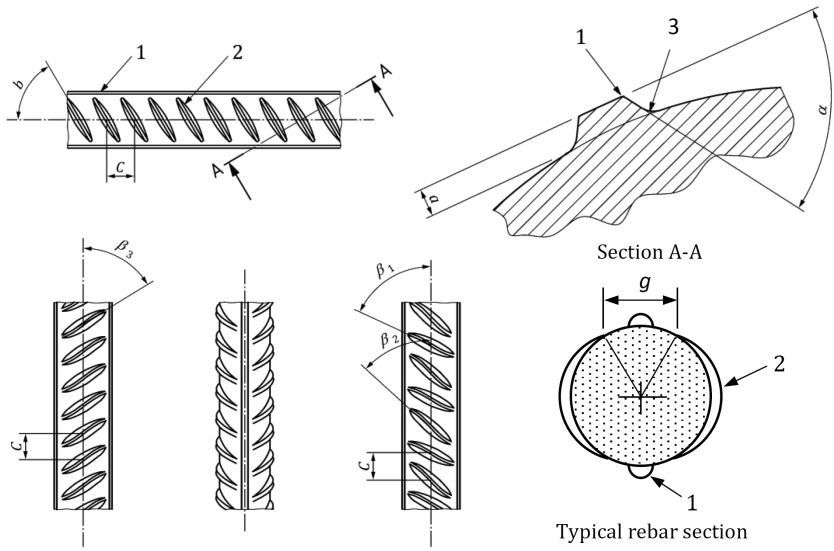

1 Longitudinal rib or deformation.
2 Transverse rib or deformation.
3 Rounded transition.
a Rib/deformation height
c Deformation spacing.
b, β_1, β_2 and β_3 Rib angles
g Gap (Chord)

FIGURE 2.2 Parameters for measurement of deformation on rebar (adapted from ISO 6935-2:2015).

2.1.4 Rebar Marking

In addition to the surface deformations, manufacturers are also required to provide information regarding the grade and type of steel on the rebar using a systematic rebar marking style. Currently, manufacturers in Bangladesh follow the ISO marking style. The requirement is that all bars should be identifiable by permanent marks that indicate the following:

a. Grade of steel.
b. Manufacturer's identification.

Further details on this topic are discussed in Chapter 6 of this book.

2.2 MECHANICAL PROPERTIES

Mechanical properties of rebar include its tensile strength characteristics and bendability. Details of these features are discussed in the subsequent articles.

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2.2.1 Tensile Properties

When a mild steel (MS) rebar is subjected to a tension force, it elongates. The force vs. elongation behavior is graphically shown in Figure 2.3. The behavior is elastic to a certain extent (until point 1 in Figure 2.3). When the tensile force exceeds the elastic limit, inelasticity and strain hardening start to occur. Mild steel rebar often shows a phenomenon called *yielding* immediately after the elastic limit. *Yielding* is characterized by a relatively large elongation of the tension sample without any appreciable change in the magnitude of the tensile force (zone 1–2 in Figure 2.3). After the elongation reaches a certain limit (point 2), the tensile force starts to increase again with an additional increase in elongation. This phenomenon is called strain hardening. As the strain hardening is continued, the rate of increase in tensile force gradually decreases, and the peak or ultimate strength is reached (point 3). Until this point, the elongation of the rebar is uniform along its length, accompanied by a small decrease in the diameter or width due to Poisson's effect. Immediately after the peak, necking starts to occur in a small zone of the rebar, typically at the mid-length position. Necking is characterized by further elongation of the rebar with a quick decrease in the diameter of the sample over a small zone and a drop in the magnitude of tensile force. The rebar eventually breaks or ruptures at the necking location with a cup-cone type cracked surface (point 4). The tensile force vs. elongation behavior, as shown in Figure 2.3, can be plotted for any rebar. However, rebar of different diameters and grade shall have different magnitudes of force and elongation, which are difficult to compare and assess. For this reason, a stress vs. strain diagram is generally plotted and studied. Such stress vs. strain diagrams typically represent material characteristics rather than a particular rebar.

The shape of a stress vs. strain diagram is similar to the force vs. elongation diagram. The typical characteristics of the stress-strain curve of a mild steel rebar are shown in Figure 2.4. When a rebar sample is tested in tension, there is an initial straight portion (line o–a in Figure 2.4) after which non-linearity starts. Point *a* is the proportional limit up to which the relation between stress and strain is linear and follows Hooke's law. The slope of this initial straight portion is called the elastic modulus or Young's modulus, *E*. After reaching the upper yield point *b*, the stress-strain diagram becomes approximately horizontal with slight fluctuation in stress as the strain increases for a while. The approximately horizontal portion is called the *yield plateau*. Having a yield plateau in the stress-strain diagram is a desirable characteristic. When a yield plateau is present, the yield stress, f_y, (or grade in ksi/MPa) is defined as the stress level corresponding to the yield plateau. After the yield plateau, the stress again increases with the increase in strain but at a diminishing rate and reaches the ultimate strength level (point *d*). Stress corresponding to this state is called *ultimate strength* or *tensile strength*, f_u. Strain corresponding to this point is referred to as the elongation at maximum force (e.m.f.) by ISO standard. When strain is further increased, a localized reduction of cross-section, often termed necking, starts accompanied by a drop in stress level. The bar eventually ruptures (point *e*) or fails physically. Strain corresponding to the failure point (point *e*) is the ultimate strain, ε_u.

Unless otherwise stated, we are referring to stress and strain as *engineering stress* and *engineering strain*, respectively. Engineering stress is calculated by dividing the

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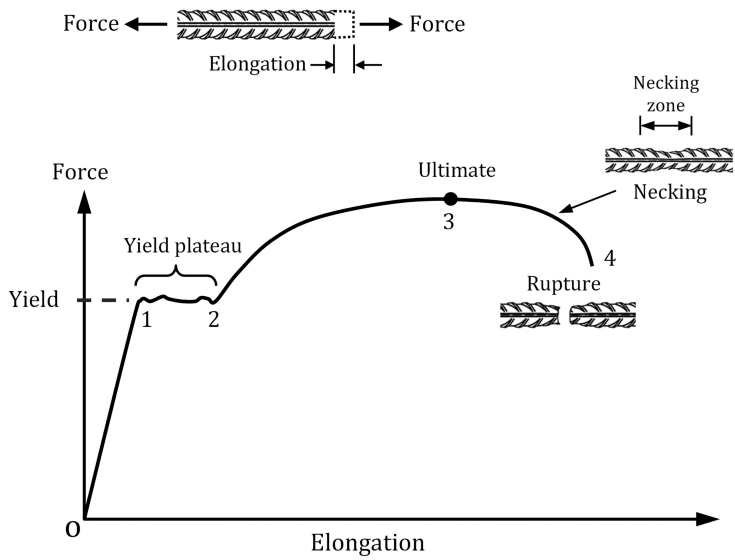

FIGURE 2.3 Tensile behavior of MS rebar.

force by the original cross-sectional area. Engineering strain is calculated on the basis of dividing the amount of elongation by the original length. True stress is calculated by dividing the force by the instantaneous cross-sectional area. The instantaneous

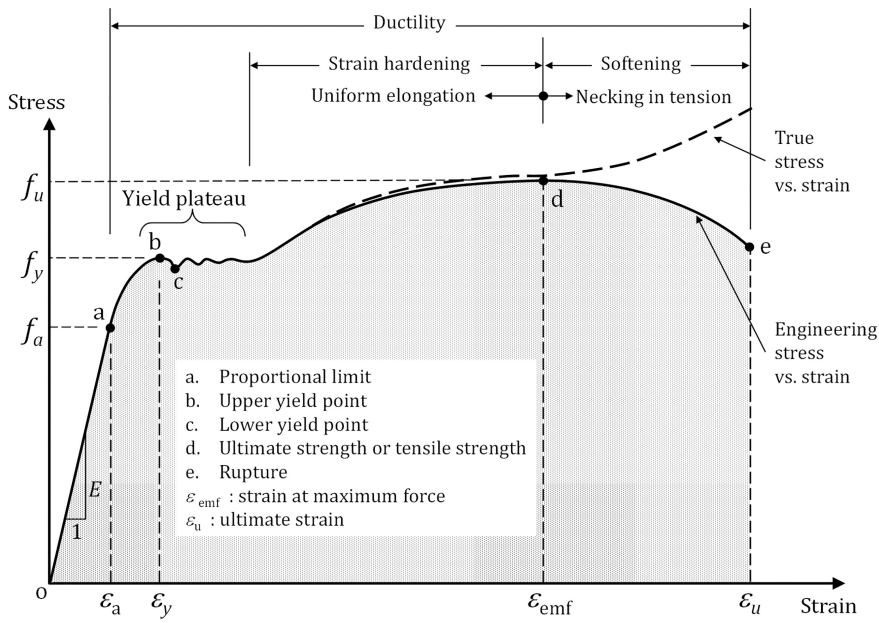

FIGURE 2.4 Typical stress-strain diagram of MS rebar showing yield plateau.

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cross-sectional area is slightly reduced until the peak (point *d*) due to the Poisson's effect, and then it is quickly reduced at the necking zone until failure. After necking, the rate of reduction of area is higher than the drop in load. Thus, if true stress vs. true strain (at the neck) is plotted, it will look like the dashed curve shown in Figure 2.4.

Sometimes, rebar samples do not show a distinct yield plateau. Instead, after the proportional limit, the stress-strain curve gradually flattens out, as shown in Figure 2.5. For such cases, ASTM and ISO, as well as other standards, specify that the yield point shall be determined on the basis of a parallel offset of the initial straight portion drawn at 0.2% strain. The stress level at the intersecting point of the offset with the stress-strain curve is defined as the yield point (point *a* in Figure 2.5). One drawback to this method of determining the yield point is that the stress-strain diagram needs to be drawn to the tension specimen to determine the yield point. Another alternative is the stress corresponding to 0.5% strain (point *b* in Figure 2.5). The advantage of this procedure is that it can be directly determined during the testing of the tension specimen without drawing the stress-strain diagram. However, this method of may not be suitable for material with a yield strength greater than 80 ksi as this would tend to result in a lower value of yield stress.

The area under the stress-strain curve is called toughness (shaded area in Figures 2.4 and 2.5). It is the total energy absorbed per unit volume to cause fracture of the rebar. If the upper limit of integration is restricted to the proportional limit, the energy absorbed per unit volume is known as the modulus of resilience. It is the recoverable part of the strain energy if stress is released from a tension sample. From Figure 2.4, the modulus of resilience is $\frac{1}{2} f_a \varepsilon_a$. Generally, the proportional limit and the yield point are very close to each other. For such a situation, the modulus of resilience may be approximated as $\frac{1}{2} f_y \varepsilon_y = \frac{1}{2} f_y^2 / E$.

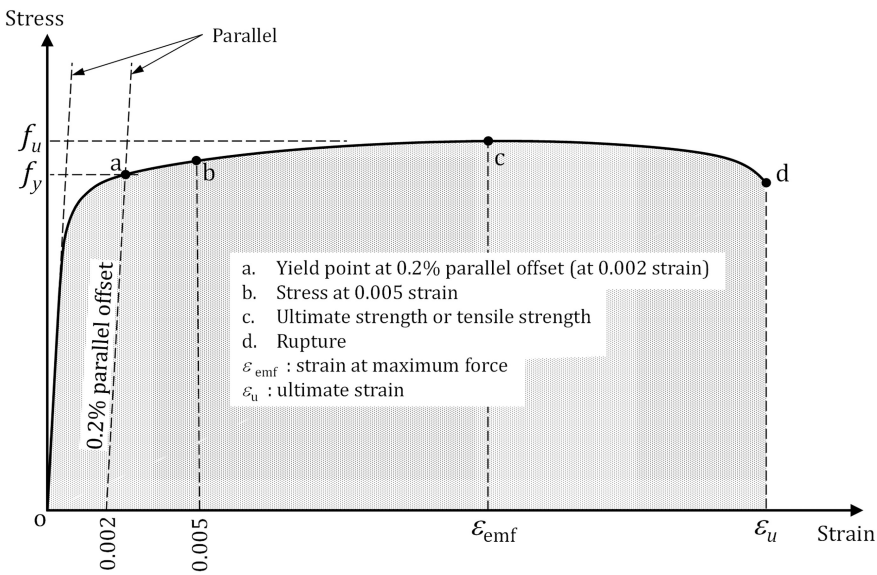

FIGURE 2.5 Typical stress-strain diagram of MS rebar without yield plateau.

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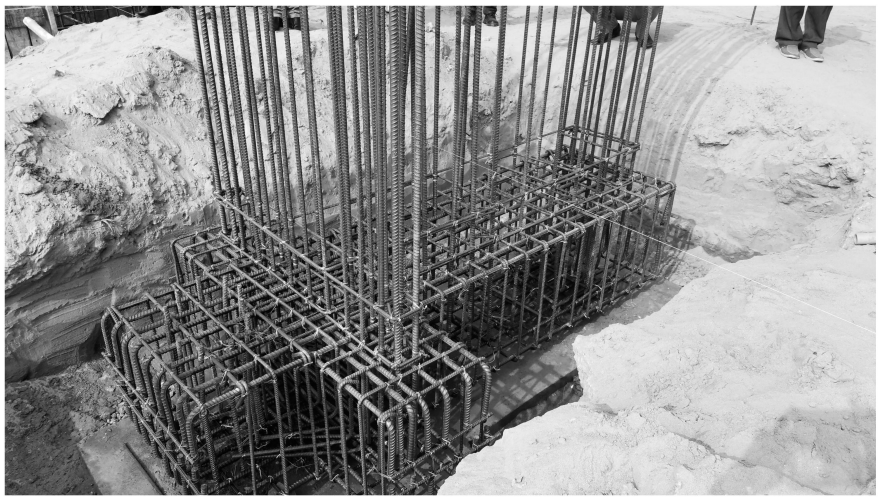
Ductility is the ability of rebar to undergo large plastic deformation before failure. The ultimate strain, ε_u, is a measure of ductility. The value of ε_u frequently exceeds a value 0.30 for 40 grade (f_y = 40 ksi or 275 MPa) MS rebar. In the rebar test report, this is generally expressed as a percentage of total elongation. Thus the ductility of this rebar would be 30%. Ductility is an important parameter in the seismic design of RC structures. Tensile strength requirements for various rebar types are given in Tables A.5 through A.8 in the Appendix.

2.2.2 Bendability

In reinforced concrete construction, steel rebar is frequently bent into desired shapes to be used in the concrete element. In fact, using rebar without bending it for hooks, cranking or ties is unthinkable. A few examples are shown in Figure 2.6.



(a) Rebar assembly in a pile cap

(b) Spirals

(c) Column tie hooks

FIGURE 2.6 Examples of bent rebar used in concrete construction.

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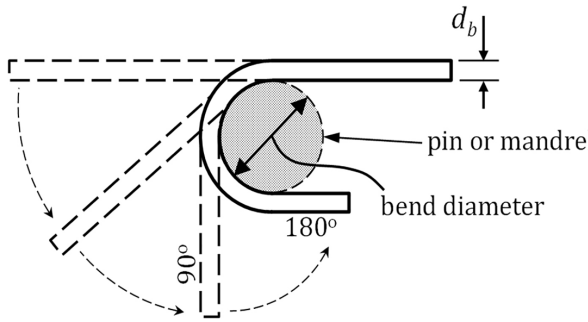

FIGURE 2.7 Bending of rebar.

Bendability is the ability of rebar to undergo permanent (plastic) bending deformation without breaking or cracking. It is an essential characteristic of good quality rebar. Bendability is also a measure of ductility, which is very important in the seismic design of concrete structures. Standards like ASTM or ISO define the bendability requirements for rebar. In these standards, bendability is defined as the ability of rebar to be bent in a certain curvature through 90° or 180° angle. The curvature is defined by a bend diameter (Figure 2.7) which is generally expressed as a multiple of the nominal diameter of the rebar.

Bendability is tested by bending rebar around a pin or mandrel of a certain diameter (Figure 2.7). As such, bend diameter is also referred to as pin diameter. Rebar with good bendability would not break, nor would it show any sign of cracking or distress upon bending around a pin of a specified diameter through the required angle. The bendability of rebar depends on its chemical composition and its tensile strength (grade). For this reason, the requirement varies among standards. The bendability requirement of various rebar is given in Tables A.9 through A.12 in the Appendix.

2.2.3 Fatigue

Fatigue is the lowering of tensile strength of a material due to repeated cyclic tensile loading with the maximum magnitude of the repeated load much lower than the yield stress of the material. Fatigue is not associated with compression. Fatigue is caused under tensile loading by progressive and localized damage in the crystalline structure of the material. When the magnitude of the repeated load is above a threshold value, microscopic cracks begin to form at edge dislocation in the crystalline structure of the material and at other stress concentrating locations. As the fluctuating loading is repeated for a few thousand or million cycles, the crack eventually grows, and the material fails. It is to be understood that fatigue is associated with fluctuating tensile loading for a large number (from a few thousand to a few million) of loading cycles. As such, fatigue is generally referred to as high cycle fatigue. In the high cycle fatigue test, the maximum stress level is generally below 60% of the yield stress.

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American rebar standards ASTM A615, A706 or A1035 do not specify any provision for fatigue testing of rebar. Testing for fatigue properties of steel is covered in ASTM E466. The European ISO 6935-2 standard specifies fatigue (high cycle) as a material parameter. When a high cycle fatigue test is performed, the number of cycles is generally specified as 5.0 million. The maximum stress level generally varies between 185 MPa to 250 MPa (36 ksi), while the ratio of minimum and maximum stress is maintained at 0.2.

Experimental evidence (Helgason *et al.* 1976; Hognestad 1961, 1962) shows that if the tensile stress level remains below approximately 23 ksi (158 MPa), then the strength of the rebar is not significantly affected by fatigue regardless of the number of cycles of loading. Under service load conditions, the stress level in rebar in most elements of concrete structures generally remains at or below this level when Grade 40 or Grade 60 rebar is used. Experimental evidence also shows that rebar yield stress in a concrete structure is not appreciably lowered by fatigue regardless of the stress level when the number of loading cycles is not more than 50,000, which is approximately equivalent to three cycles of loading per day for 45 years. For this reason, fatigue of rebar is of no concern to the design of reinforced concrete buildings and similar structures. For seismic design, fatigue (high cycle fatigue) is not an important issue. In AISC standard (ANSI/AISC 360-05), fatigue is defined as *"Limit state of crack initiation and growth resulting from repeated application of live loads."* On the other hand, *static load* is defined as "*Not subject to significant fatigue stresses. Gravity, wind and seismic loadings are considered to be static loadings."*

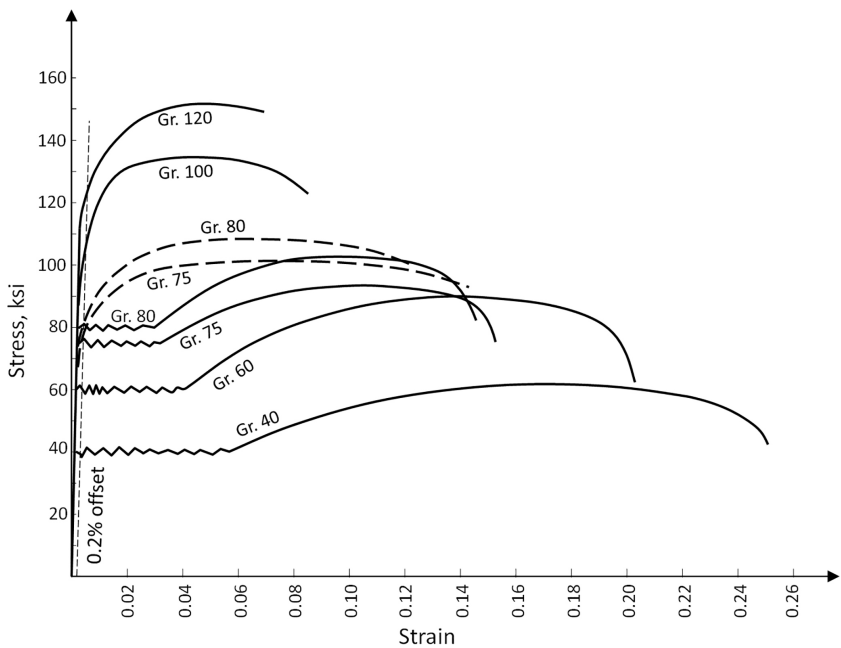

FIGURE 2.8 Comparative stress-strain diagram of different grades of rebar.

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These code definitions clearly rule out fatigue from seismic design considerations of buildings. For this reason, the ACI 318-19 code and other leading building codes do not emphasize fatigue (high cycle fatigue) as a design parameter for seismic design of buildings.

For bridges and other structures where the number of cycles of repeated loading (live load) at a relatively higher stress level (but below the yield stress) frequently exceeds 1.0 million in a short span of time, fatigue is an important design consideration. American design standards like AASHTO LRFD Bridge Design Specifications (AASHTO 2010) do provide provisions for fatigue design of RC members for live load.

2.3 VARIATION OF THE SHAPE OF THE STRESS-STRAIN DIAGRAM WITH STRENGTH

A comparison of stress-strain diagrams with different grades of steel is shown in Figure 2.8. It can be observed that generally, Grade 40 and Grade 60 rebar show a well-defined yield plateau. Most of Grade 75 (520 MPa) or 72.5 (500 MPa) rebar manufactured in Bangladesh also shows a well-defined yield plateau. However, some Grade 72.5, 75 and Grade 80 rebar may not show the yield plateau. Instead, these rebars may follow the curves shown by the dashed lines. Higher grades such as Grade 100 and Grade 120 usually do not show any yield plateau, for which the 0.2% offset method invariably applies for yield point determination. Comparing the stress-strain diagrams in Figure 2.8, it can be observed that Grade 40 shows the highest ductility, followed by Grade 60 rebar. The ductility of grades higher than Grade 60 is generally significantly low. From a seismic design point of view, ductility is an essential characteristic that must be exhibited by rebar used in concrete construction. Another characteristic of the stress-strain diagram which may be considered important is the toughness or energy absorbed before the rupture, which is defined as the area under the stress-strain diagram (shaded area in Figure 2.4). High toughness is an essential characteristic for the earthquake-resistant design of concrete structures.

2.4 WHY YIELD PLATEAU IS DESIRABLE

Almost all concrete design codes, except the Japanese one, permits the use of mild steel rebar which does not exhibit any yield plateau in the stress-strain behavior. In such a case, the 0.2% offset method (Figure 2.5) is used to determine the yield strength. The behavior of a concrete flexural member with rebar without yield plateau will be different from one with rebar having yield plateau. As an example, the stress-strain diagram for a pair of Grade 75 rebar having the same ultimate strength of 94 ksi is shown in Figure 2.9 – one with yield plateau (solid curve) and the other one without yield plateau (dashed curve). Let us assume that visible cracks occur at strain value of 0.015. Once a crack is visible, the occupants will be alerted. When the rebar has yield plateau, this corresponds to point *a* in Figure 2.9. We see that there is still a reserve strength of about 19 ksi before the rebar reaches its ultimate strength (point *c*). If the rebar does not have yield plateau, the same cracking strain (or visible crack) corresponds to point *b* in Figure 2.9 where steel stress is 88 ksi. The reserve strength corresponding to

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this point is only 6 ksi. We thus see that for the same amount of visual distress (same crack width or same cracking strain), rebar with yield plateau provides higher reserve strength, i.e. provides enough time and better opportunity to take remedial measures. On the other hand, when the crack is visible in the case of rebar without yield plateau (obviously, it will be at a higher load), the reserve strength before reaching the ultimate strength is low, allowing less time and limited opportunity to take remedial measures. Thus, rebar with a yield plateau is safer and more desirable.

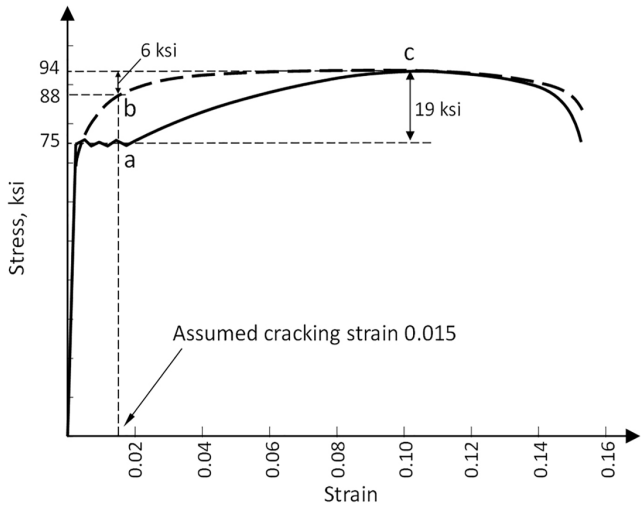

FIGURE 2.9 Significance of yield plateau.

2.5 BRIEF METALLURGY OF MILD STEEL

The process of making structural steel from iron ore or from scrap steel is a vast topic; covering all aspects of it is beyond the scope of this book. Only a very brief introduction to the steel-making process is presented here to facilitate a glimpse of the topic for the reader. For further information on this topic, the reader is encouraged to read any standard textbook on steel-making, including the one written by Ghosh and Chatterjee (2008) or the one written by Tupkary and Tupkary (2018).

Iron ore, coal and lime are mixed and heated in a blast furnace with a capacity of 100 to 400 tons at the beginning of the steel-making process. Lime serves as a flux and creates a layer of protection on top of the hot, molten metal. Basic Oxygen Furnace (BOS) or Electric Arc Furnace (EAF) processes are used in contemporary primary steel-making. Both are based on the original Bessemer steel-making process. While scrap steel is being added to the converter, oxygen is blown through the molten iron during the BOS process. The carbon content is lowered to a maximum of 1.5% as a result. In the process of making steel by recycling, scrap steel is fed into an electric arc furnace and passed through incredibly powerful electric arcs and is processed as described above. High-quality steel can be produced in this manner.

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Pure iron is soft and generally useless as an engineering material; adding small amounts of carbon is the primary method for strengthening it and transforming it into steel with desirable structural properties such as strength and ductility. Changing the carbon content of steel is the most common way to alter its mechanical properties (Britannica 2022).

Heat treatment, such as accelerating the rate of the cooling process, enables additional modifications in the properties of steel. By quenching the steel at approximately 1,000°C per minute, carbide formation is completely suppressed, and a new microstructure called martensite is created. It is the hardest and most fragile form of steel. Tempering martensitic steel reduces its hardness and brittleness, resulting in a steel that is strong and ductile. Quench-and-temper heat treatments are applied at a variety of cooling rates, holding times and temperatures; these are an essential means of controlling the properties of steel (Britannica 2022).

As a third option, alloying elements other than carbon can be used to produce steel with properties that are not possible in plain carbon steel. Adding these elements alters the transition points between ferrite and austenite and forms intermetallic compounds like carbides and nitride. Using computer applications, engineers can control the microstructure and properties of steel during the processes of hot rolling, heat treating and cold forming (Britannica 2022).

The molten steel is poured into a mold once it has the desired carbon content and other elements that improve its engineering characteristics. The steel is thus cast and shaped before being allowed to solidify. The metal is then cut into the desired shapes, such as slabs, blooms and billets. The final product still needs to have a good surface quality without casting flaws. Hot rolling is frequently used in the primary forming process for this purpose (Britannica 2022).

Although the production of mild steel products is nearly finished by the end of the aforementioned hot-rolling process, they are typically formed a second time to enhance their mechanical properties in accordance with application requirements. Its final shape and characteristics are the result of secondary forming. Cold rolling, machining, coating, tempering or thermo-mechanical treatment and surface treatment are a few of the final forming techniques.

2.6 CHEMICAL COMPOSITION OF STEEL

For an engineer designing an RC structure, the chemical properties of steel rebar are probably not as important as its mechanical properties, such as yield strength and elongation, etc. However, it must be kept in mind that the mechanical and durability characteristics of steel rebar are controlled by its chemical composition. It is the alloy composition that is mostly responsible for controlling the important engineering properties of steel rebar. The strength and ductility of steel are the most important engineering properties for the safe structural design of buildings. These are significantly affected by chemical composition, heat treatment and the method of steel manufacturing. Among many chemical properties, a structural engineer should be concerned with the weldability and corrosion resistance characteristics of steel rebar.

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Different steel grades that are either published, registered or standardized around the world have different chemical compositions, and various countries have created unique numbering or identification systems to categorize the enormous number of alloys. Additionally, there is a huge variety of options available to the user of steel due to the various heat treatments, microstructures, cold-forming processes, shapes and surface finishes. Despite such diversification of steel grades, structural steels can be divided into a few major groups fairly accurately based on their chemical composition, applications, shapes and surface characteristics (Britannica 2022).

Steels can be divided into three major classes based on chemical composition:

a. Carbon steel,
b. Low-alloy steel, and
c. High-alloy steel.

There is a trace amount of incidental materials left over from the steel-making process in every steel. These include phosphorus and sulfur taken up from the ore and fuel in the blast furnace, as well as manganese, silicon or aluminum from the deoxidation process carried out in the ladle. Scrap used in the steel-making furnace introduces copper and other metals, known as residuals. These substances are not alloys because they typically make up less than 1% of the steel as a whole (Britannica 2022).

2.6.1 Carbon Steel

About 90% of the steel produced worldwide is carbon steel, making it the most produced and used type of steel. They are typically divided into high-carbon steels, with carbon content above 0.5%, medium-carbon steels, with a carbon content between 0.2 and 0.49%, low-carbon steels, with a carbon content between 0.05 and 0.19%, extra-low-carbon steels, with a carbon content between 0.015 and 0.05%, and ultralow-carbon steels, with carbon content below 0.015%. Carbon steels must also contain less than 1.65% manganese, 0.6% silicon, and 0.6% copper, with a maximum combined content of 2% for all of these additional elements (Britannica 2022).

2.6.2 Low-Alloy Steel

Low-alloy steels contain alloying elements up to 8%; any concentration above this is referred to as high-alloy steel. Other than carbon, there are about 20 alloying elements. These include nitrogen, sulfur, copper, boron, lead, tellurium, selenium, manganese, silicon, aluminum, nickel, chromium, cobalt, molybdenum, vanadium, tungsten, titanium, niobium and zirconium. To achieve specific properties, several of these are frequently added simultaneously (Britannica 2022).

2.6.3 High-Alloy Steel

High-alloy steel is a type of steel that has a larger proportion of alloying elements compared to those present in carbon steel or traditional low-alloy steel. This kind

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of steel possesses special micro-structures and requires heat treatment that differs considerably from those of plain carbon steels. Generally, the portion of alloying elements constitutes at least 8% of the mass. The mechanical qualities and corrosion resistance are greatly improved by these extra ingredients. Metals like chromium, nickel and molybdenum, etc. are used as alloying elements. High-alloy steel is a favored material in many demanding applications because of its reputation for strength, durability and ability to endure harsh conditions. A typical example of high-alloy steel is the stainless-steel which contains at least 10.5% chromium as the main alloying element. Further detail on the use of stainless-steel as concrete reinforcing material can be found in Chapter 8.

2.7 CHEMICAL COMPOSITION OF REBAR MATERIAL

According to ASTM standards, the chemical analysis of each heat shall be determined in accordance with ASTM A751 *Standard Test Methods, Practices, and Terminology for Chemical Analysis of Steel Products*. The manufacturer must conduct the analysis on test samples taken, preferably during the heat pouring process.

2.7.1 ASTM A615

Rebar conforming to ASTM A615 is carbon steel. For such steel, the percentages of carbon, manganese, phosphorus and sulfur shall be determined for quality control purposes. The phosphorus content thus determined shall not exceed 0.06%. A product check for phosphorus made by the purchaser shall not exceed 0.075%. ASTM A615 specification does not provide any further details about the required composition. However, in most cases, the overall composition of A615 steel is as follows:

Element	Content
Carbon (C)	0.20~0.44%
Phosphorus (P)	0.06% max.
Sulfur (S)	0.05% max.
Copper (Cu)	0.5% max.
Nitrogen (N)	0.012% max.
Manganese (Mn)	0.60~1.40%
Molybdenum (Mo)	0.05% max.
Vanadium (V)	0.01~0.06%
Silicon (Si)	0.10~0.40%
Chromium (Cr)	0.3% max.

2.7.2 ASTM A706

Rebar conforming to ASTM A706 belongs to low-alloy type steel. The general composition of A706 steel is shown in the following table.

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Element	Content
Carbon (C)	0.24–0.30%
Phosphorus (P)	0.01–0.02%
Sulphur (S)	0.02–0.04%
Copper (Cu)	0.20–0.40%
Nitrogen (N)	0.01–0.015%
Manganese (Mn)	1.00–1.25%
Molybdenum (Mo)	0.03–0.06%
Vanadium (V)	0.02–0.05%
Silicon (Si)	0.15–0.30%
Chromium (Cr)	0.10–0.20%
Nickel (Ni)	0.10–0.20%

The percentages of carbon, manganese, phosphorus, sulfur, silicon, copper, nickel, chromium, molybdenum and vanadium shall be determined for quality control purposes. The chemical composition, as shown by heat analysis, shall be limited by the following:

Element	Max, %
Carbon	0.30
Manganese	1.50
Phosphorus	0.035
Sulfur	0.045
Silicon	0.50

The manufacturer shall select and use alloying elements combined with carbon, phosphorus and sulfur to produce the required mechanical properties. Manganese, silicon, copper, nickel, chromium, molybdenum, vanadium, columbium, titanium and zirconium are common elements. The heat analysis must be performed in such a way that the carbon equivalent (CE) does not exceed 0.55%, as calculated by the formula given in Equation 2.2.

$$CE = \%\,C + \frac{\%\,Mn}{6} + \frac{\%\,Cu}{40} + \frac{\%\,Ni}{20} + \frac{\%\,Cr}{10} - \frac{\%\,Mo}{50} - \frac{\%\,V}{10} \leq 0.55\% \qquad 2.2$$

A product check analysis made by the purchaser shall not exceed the following percentages:

Element	Max, %
Carbon	0.33
Manganese	1.56
Phosphorus	0.043
Sulfur	0.053
Silicon	0.55

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2.7.3 ISO 6935-2

In order to determine chemical composition, sampling and preparation of samples shall be in accordance with ISO 14284. Chemical analysis representative of the cast shall be determined by the manufacturer in accordance with its own procedures. For common rebar grades, the chemical composition of the steel, as determined by cast analysis, shall conform to the following table.

Steel grade	C	Si	Mn	P	S	N	CE
B400CWR B500CWR	0.22	0.60	1.60	0.050	0.050	0.012	0.50
B400DWR	0.29	0.55	1.80	0.040	0.040	0.012	0.56
B420DWR	0.30	0.55	1.50	0.040	0.040	0.012	0.56
B500DWR	0.32	0.55	1.80	0.040	0.040	0.012	0.61

The carbon equivalent, CE, can be calculated according to the formula as given in Equation 2.3

$$CE = C + \frac{Mn}{6} + \frac{(Cr + V + Mo)}{5} + \frac{(Cu + Ni)}{15} \qquad 2.3$$

Where C, Mn, Cr, V, Mo, Cu and Ni are mass fractions expressed as percentages of the steel's respective chemical elements. The permissible deviation of the product analysis from the cast analysis is shown in the table below.

Elements	Specified maximum value in cast analysis in table %	Permissible deviation in product analysis from the specified limits of the cast analysis in table %
C	≤0.25	+0.02
	>0.25	+0.03
Si	≤0.60	+0.05
Mn	≤1.65	+0.06
	>1.65	+0.08
P	≤0.05	+0.008
	>0.05	+0.010
S	≤0.05	+0.008
	>0.05	+0.010
N	≤0.012	+0.002

2.7.4 ASTM A1035

ASTM A1035 was first published in 2004 to provide standard specifications for a new type of low-carbon chromium steel with a yield strength of 100 and 120 ksi (690

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and 830 MPa). This kind of steel was first commercially produced in 2001, which was the result of two decades of research by Dr Gareth Thomas of the University of California, Berkeley (ECT Team 2007). This steel is also termed MMFX or MMFX2 steel, which stands for Martensitic Microcomposite Formable Steel. It is highly resistant to corrosion and, in many cases, outperforms conventional carbon steel in terms of strength, energy absorption, toughness, brittleness, ductility and formability. MMFX steel's superior corrosion resistance is a result of its patented and proprietary microstructure, which is formed during the manufacturing process. Conventional carbon steels form a matrix of chemically dissimilar materials – carbide and ferrite. These carbides are strong yet brittle – immovable at the grain boundaries. In a humid environment, a battery-like effect occurs between the carbides and the ferrites which destroys the steel from the inside out. This effect is the primary corrosion initiator that drives the corrosion reaction. MMFX's patented proprietary steel technology forms a matrix that is almost carbide-free (MMFX 2011).

MMFX 2 steel is a low-carbon, chromium alloy steel that is produced as part of a controlled-rolling process, i.e. rolling steel within a well-defined temperature range and cooled at a specific rate. The combination of MMFX 2 steel's chemical composition and manufacturing production process produces a steel that has a completely different structure at the atomic scale which is similar to a laminated lath microstructure resembling layers. Steel made using MMFX nanotechnology minimizes the formation of micro-galvanic cells (the driving force behind corrosion). MMFX's layering effect provides superior strength, ductility, toughness and corrosion resistance. Because of its superior strength, MMFX offers the possibility of reducing construction expenses by lowering the amount of materials used. In addition, MMFX bars make the placement of concrete easier as a result of greater bar spacing. In addition, MMFX's superior corrosion resistance enables the reduction of concrete cover in certain structures in corrosive environments. The chemical composition of MMFX steel specified by ASTM A1035 is given in the following table (MMFX 2011).

	Composition max. %						
Alloy type	**C**	**Cr**	**Mn**	**N**	**P**	**S**	**Si**
1035 CL	0.3	2.0–3.9	1.5	0.05	0.035	0.045	0.5
1035 CM	0.2	4.0–7.9	1.5	0.05	0.035	0.045	0.5
1035 CS	0.15	8.0–10.9	1.5	0.05	0.035	0.045	0.5

2.8 TMT REBARS

TMT is a metallurgical process that stands for thermo-mechanically treated. TMT rebars have a composite cross-section – a softer inner core surrounded by a tough and hard outer core. At the end of the rolling process, rebars are quickly cooled in a quenching box as they exit the final rolling mill stand. Quenching prevents unwanted processes like phase transformations from occurring. These undesirable reactions

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are less likely to occur if they are confined to a shorter period of time. Also, the rapid temperature change hardens the steel bar's outer layer, making it stronger and more durable. As a result, the rebar's outer surface becomes martensite, a hard form of steel, and shrinks, creating internal pressure that aids in the crystallization process of the inner core. The surface of the quenched bar cools and hardens as a result of this process, but the core remains hot. A temperature gradient is formed through the quenched bar's cross-section after it leaves the quenching box. As a consequence, heat flows from the core to the outer surface, which is cooler. This causes the outer martensitic layer to be correctly tempered into tempered martensite and the formation of a martensite-bainite intermediate ring (a plate-like microstructure). The core is still austenitic at this stage (a cubical crystalline structure commonly referred to as gamma-phase iron).

The bars are then subjected to atmospheric cooling to balance the temperature differences between the soft inner core and the hardened exterior after the self-tempering process. The austenitic core of the bars changes into a ductile ferrite-pearlite structure once it has completely cooled. The final product's cross-section shows a variation in its crystal microstructure, with a tough, strong, tempered martensite in the outermost layer, a martensite and bainite intermediate layer, and a refined, tough and ductile ferrite and pearlite core (Figure 2.10).

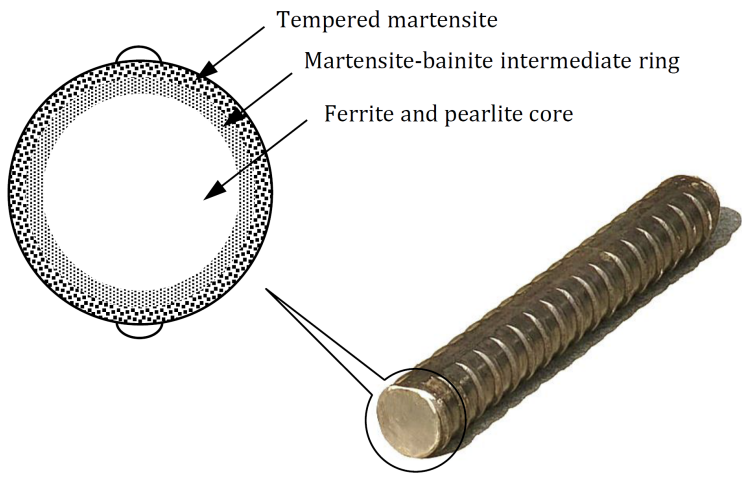

FIGURE 2.10 Composite structure of TMT rebar.

By utilizing the TMT process, higher strength rebar is manufactured from raw materials of lower strength steel. For example, typically, the raw materials of Grade 275 steel are used to obtain Grade 500 TMT rebars. This has the beneficial effect of lowering the production cost while still achieving the desired structural characteristics of the rebar. Currently, the majority of reinforcement products available in the market are TMT products.

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2.9 CORROSION OF STEEL IN CONCRETE

The integrity of a reinforced concrete structure is dependent on the integrity of the steel, the concrete and the bond between them. In addition to causing the concrete component to fail, deterioration of any one of these components can lead to the failure of another component. Cracks in the concrete allow aggressive agents from the environment to penetrate the concrete and reach the steel, causing corrosion of the steel. The development of bulky corrosion products causes cracking and spalling of the concrete. The corrosion of steel in concrete is a vast topic. Covering every aspect of it in detail is beyond the scope of this book. Only a brief introduction is presented here to facilitate a glimpse of the issue for the reader. For further information, the reader is encouraged to read the textbooks of Chess and Green (2020) and Cherry and Green (2021). Much of the information presented in subsequent paragraphs are sourced from these publications.

2.9.1 Corrosion of MS Rebar and Rust Formation

In the presence of moisture or a wet environment, an anodic reaction is established, as shown below, that involves the dissolution of iron.

$$2Fe \rightarrow 2Fe^{++} + 4e^{-}$$

The electrons generated by this anodic reaction travel through the metal and the film to take part in the cathodic reaction in the presence of oxygen and moisture.

$$O_2 + 2H_2O + 4e^{-} \rightarrow 4OH^{-}$$

This occurs throughout the unbroken passive film. Hydroxyl and ferrous ions travel through the water found in the concrete's pores to complete the circuit. Since the hydroxyl ions are more mobile than the ferrous ions, they encounter each other close to the anodic area, where rust-forming reactions occur as follows:

$$2Fe^{++} + 4(OH)^{-} \rightarrow 2Fe(OH)_2$$

$$3Fe(OH)_2 + H_2O \rightarrow Fe(OH)_3 + H_2O$$

$$2Fe(OH)_3 \rightarrow Fe_2O_3 + 3H_2O \text{ [red rust]}$$

$$2Fe(OH)_3 + Fe(OH)_2 \rightarrow Fe_3O_4 + 4H_2O \text{ [black rust]}$$

The above equations dictate that the formation of corrosion products are only possible in the presence of water and oxygen. The rust products have three to six times the volume of corresponding iron taking part in the rust formation. The relative volume of different rust products (Marcotte 2001) are shown in Figure 2.11. When the rusting iron is confined inside the concrete, due to their increased volume, the rust products exert outward pressure or bursting stress in the concrete. When this bursting

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stress exceeds the confining capacity of concrete, the concrete gets cracked, leading to delamination and then spalling. Rust stains may or may not occur concurrently with cracking or as a precursor to delamination and spalling. Reinforcement section loss, bond loss and anchorage loss also occur as a result of corrosion. Research has shown that (Kolio *et al.* 2015) when corrosion is induced by carbonation, the corrosion products such as α-FeOOH and γ-FeOOH can have a volume roughly 3.0 times the volume of iron. Hydrated hematite (α-$Fe_2O_3.3H_2O$) (red rust) induced cracking, delamination and spalling of cover concrete can be frequently seen in deteriorated reinforced concrete structures and buildings as this corrosion product has a volume as much as six times that of iron.

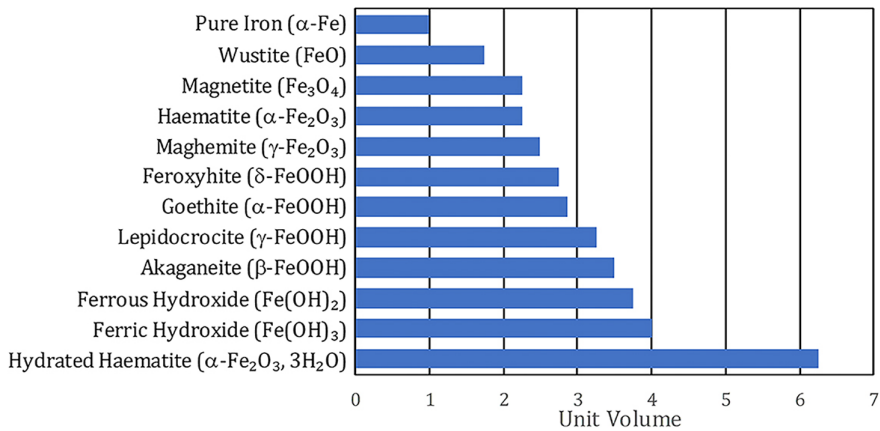

FIGURE 2.11 Relative volumes of rust products (Marcotte 2001).

2.9.2 Protection of Steel by Concrete

Reinforced concrete structures and buildings can have a service life of many decades when properly designed, built and maintained. Reinforcing steel is generally well protected from corrosion by concrete. At the time of casting, the steel reinforcement surface forms a stable, tightly adhering, thin protective iron oxide passive film in the highly alkaline environment of concrete. This passive film protects the reinforcement from corrosion. The hydration reaction of cement compounds causes the pore water surrounding the steel in concrete to have a high pH (in the range of 12–14). This leads to the formation of a dense gamma ferric oxide (γ-Fe_2O_3, maghemite) passive film which completely covers the reinforcement and protects it. This is formed by the anodic reaction:

$$4Fe\ (s) + 12OH^-\ (aq) \rightarrow 2\gamma Fe_2O_3\ (s) + 6H_2O\ (l) + 12e^-$$

After the formation of the passive film, the reaction continues at the metal/oxide interface as a consequence of hydroxyl ions diffusing through the film of iron oxide. This cathodic reaction proceeds slowly after the formation of the film:

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$$3O_2\,(g) + 6H_2O\,(l) + 12e^- \rightarrow 12OH^-\,(aq)$$

The reaction occurs on the passive film's surface. In order for the cathodic process to work, oxygen molecules must diffuse through the concrete to the surface of the passive film, where they are converted to hydroxyl ions. The hydroxyl ions produced by the cathodic reaction are transported ionically through the passive film toward the metal/film interface and electronically toward the film/electrolyte interface to complete the electrical circuit. In this state, corrosion is slowly happening in accordance with the transformation of iron into iron oxide. Passive (leakage) corrosion current density measured for steel reinforcement in undamaged concrete is typically of the order of 0.1 μA/cm^2 (Cherry and Green 2021). This corrosion rate is about 1 mm per 1,000 years, so the steel in concrete may be relatively stable against corrosion. This state of steel is described as passive or as being protected by a passive film.

The thickness of the passive film is generally of the order of 6 to 9 nanometers. As long as the integrity of the concrete surrounding the rebar is intact and the concrete is dense and well compacted, the passive film remains impervious, and the rebar remains well protected. This protective film is susceptible to attack by aggressive environmental agents. The most significant of these aggressive agents are chloride ions, which effectively create holes in the film. They also generate carbon dioxide gas which dissolves in the pore water of concrete to produce an acidic solution. This acidic solution lowers the pH of concrete to below 10, neutralizes the alkalis and thereby renders the passive film unstable. There are three primary ways for aggressive agents to reach the metal surface: through cracks in the concrete, through pores in the cement phase or by permeating through the concrete under pressure (Cherry and Green 2021).

2.9.3 Chloride Induced Corrosion

The presence of chlorides is the most common reason for the initiation of corrosion of steel reinforcement in concrete. Chlorides may originate from marine environments, deicing and anti-icing salts, industrial brines, contamination or chloride-containing additives. Due to the difficulty of examining the process on an atomic scale in the extremely thin passive layers, the precise mechanism of breakdown of the passive film by chlorides has not been well defined. Recent research (ACI 222R-19) indicates that changes in surface and local critical chemical conditions enable charge exchange pathways that eliminate the passive film's blocking ability, thereby depassivating the surface. Studies (ACI 222R-19) have demonstrated that chlorides reduce the proportion of Fe^{+2} oxides relative to Fe^{+3} oxides, resulting in depassivation of the steel surface. The end result is that active corrosion can occur at these sites, and once it begins, it advances autocatalytically or in a self-feeding manner. The chlorides and ferrous ions combine to create a soluble complex that diffuses away from the anodic site. When the complex reaches an area with a high pH, it decomposes, precipitating an insoluble iron hydroxide and releasing the chloride to remove additional iron from the steel reinforcement.

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The concentration of chloride ions that needs to arrive at the steel surface to destroy passivity is often termed the *threshold concentration*, *critical chloride level/concentration* or *critical chloride content*. There is no universally accepted threshold concentration due to many factors (Cherry and Green 2021), such as a) moisture content, b) availability of oxygen, c) surface condition of the steel, d) concentration of OH^- ions, e) water-cement ratio, f) electrical resistivity of concrete, g) degree of hydration, h) chemical composition of steel, i) temperature, j) source of chloride, k) presence of inhibitors and l) electrode potential of steel, etc. For these reasons, the chloride threshold levels reported by past researchers have a wide variation (Cherry and Green 2021). In a comprehensive study spanning over a decade conducted by AASHTO in its SHRP2 program (Pease *et al.* 2019), it was reported that for ordinary carbon steel reinforcement (e.g. ASTM A615, A706 etc.), the chloride threshold value ranges from 0.13 to 2.16% by weight of cement, the average being 0.68 with a standard deviation of 0.47. Such a wide variation in chloride threshold value suggests that it should be determined for each and every project experimentally for the actual systems of interest, considering both the concrete and the rebars with their actual properties.

2.9.4 Carbonation Induced Corrosion

The alkaline environment of concrete provides steel reinforcement with passivity, but this passivity can be lost if the pH of the concrete pore solution drops due to carbonation, damaging the passive film. When gaseous carbon dioxide from the air dissolves in concrete pore water and seeps inside, as well as when the concrete surface is exposed to water or soil that contains dissolved carbon dioxide, carbonation may happen. By reaction, carbon dioxide dissolves in pore water to create carbonic acid (Cherry and Green 2021):

$$CO_2\ (g) + H_2O\ (l) \rightarrow H_2CO_3\ (aq)$$

The decomposition of carbonic acid into hydrogen and bicarbonate ions is possible in a moist environment. The neutral, insoluble $CaCO_3$ is produced when carbonic acid reacts with $Ca(OH)_2$ in the solution that exists in the pores of the hardened cement paste. The general reaction is as follows:

$$Ca(OH)_2\ (aq) + H_2CO_3\ (aq) \rightarrow CaCO_3\ (s) + 2H_2O\ (l)$$

The end result is a decrease in the pore water's alkalinity. High alkalinity is necessary to maintain a passive film on reinforcing steel. Even though there is a pH buffer between CaO and $Ca(OH)_2$ that maintains the pH at about 12.5, CO_2 reacts until the buffer is consumed, at which point the pH will decrease to levels that are no longer protective of steel.

At pH levels greater than 10, the iron oxide passive film that forms on steel under alkaline conditions is stable. The passive film is dissolved because the carbonation of concrete lowers the pH of the pore water to below 9. The pH environment around

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the steel drops to below 9, which causes corrosion to start and spread. Concrete that has been carbonated develops general corrosion in the areas where anodic and cathodic reactions are occurring right next to one another (microcells or minicells), resulting in the uniform dissolution of steel (iron) across the entire surface. However, the steel reinforcement may experience localized carbonation induced corrosion at cracks, concrete flaws, low cover areas, etc. of reinforced concrete elements (Cherry and Green 2021).

2.9.5 Leaching Induced Corrosion

If the pH of the concrete pore solution decreases due to the leaching of $Ca(OH)_2$ (and NaOH and KOH), the alkaline environment of concrete that provides steel reinforcement with passivity is lost. Leaching from concrete can cause the pH to drop below 10, which will cause steel reinforcement to corrode. A saturated solution of calcium hydroxide that is in equilibrium with the calcium silicate hydrates that make up the cement gel can be found in the capillary pores of a hardened cement paste. If soft water can pass through the concrete (for example, through a joint, crack or other defects), it can leach free calcium hydroxide from the hardened cement gel, diluting the pore water and lowering the pH. Leached calcium hydroxide reacts with atmospheric carbon dioxide to produce deposits of white calcium carbonate on the concrete's surface. These calcium carbonate deposits may appear as stalactites or as severe efflorescence. Where it occurs at concrete joints, cracks and defects etc., leaching induced corrosion of steel reinforcement can be localized and thus be more serious (Cherry and Green 2021).

2.9.6 Stray and Interference Current Induced Corrosion

Corrosion of steel-reinforced, prestressed and post-tensioned concrete elements can be caused by stray electrical currents, which typically originate from electric traction systems. Ground currents are also particularly effective at causing corrosion in long reinforced concrete structures, such as concrete pipelines, tunnels and retaining walls, because the potentials that can accumulate in the soil between the various portions of the structures are proportionally greater. Even though the driving force for corrosion is greater, the resistance between the cathode and anode does not increase proportionally, resulting in greater corrosion currents. Schemes for cathodic protection are typically associated with interference corrosion currents. The application of remote anode impressed current cathodic protection to (for example) a buried pipeline requires the installation of anodes (anode ground-bed) at a distance from the pipeline in order to pass current from the anodes (ground-bed) to the pipeline. If a reinforced concrete structure is present within this potential field, potential gradients associated with the passage of current through the soil will cause interference corrosion of steel reinforcement. The potential loss of the reinforcing steel section due to stray and interference current corrosion can be substantial, and it may be necessary to develop design measures to mitigate such corrosion (Cherry and Green 2021).

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QUESTIONS

2-1. Explain how rebar sizes are designated in US and ISO standards.
2-2. What is the actual meaning of diameter of a deformed reinforcing bar? Explain.
2-3. How much deviation in weight is allowed for reinforcing bars in ASTM and ISO standards?
2-4. Why is surface deformation important for rebar? Write the deformation requirements according to ASTM standards.
2-5. Describe the surface deformation requirements of rebar according to ISO standard with neat sketches.
2-6. Describe, with neat sketches, the behavior of a typical mild steel rebar in tension.
2-7. Draw the typical stress-strain diagram for mild steel rebar and describe the following:
   a. Yield plateau
   b. Strain hardening and softening
   c. Proportional limit
   d. Upper end and lower end yield point
   e. Ultimate strength
   f. Rupture strength
   g. Ductility
2-8. Discuss, with neat sketches, the difference between engineering and true stress vs. strain.
2-9. Discuss, with neat sketches, how you would determine the yield strength of rebar whose stress vs. strain diagram does not show any yield plateau.
2-10. What is bendability of rebar? How is it determined?
2-11. What is fatigue, and why is it important for rebar? Discuss the stress level and number of cycles specified in a fatigue test.
2-12. Discuss why fatigue properties of rebar are important for bridges but not for buildings?
2-13. Draw and discuss comparative stress-strain diagrams for different grades of rebar. What is the characteristic difference of the stress-strain diagrams of lower and higher grades of steel?
2-14. Discuss why a rebar having yield plateau in the stress-strain diagram is preferable?
2-15. Provide a brief overview of the process for making structural steel.
2-16. What are the major classes of steel based on chemical composition? Briefly discuss the classes.
2-17. What are the characteristics of ASTM A1035 steel and how does it differ from other conventional carbon steel?
2-18. What is TMT? With a neat sketch, describe the metallurgical process used to obtain a TMT rebar.

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2-19. Briefly describe the chemical process of rust formation and how rust products deteriorate concrete.
2-20. Explain how concrete provides protection to steel.
2-21. Write short notes on corrosion caused by the following factors:
   a. Chloride
   b. Carbonation
   c. Leaching
   d. Stray and Interference Currents

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3 Testing for Engineering Properties of Rebar

3.1 GENERAL

Testing rebar to determine its various engineering properties is necessary for establishing its quality and maintaining construction standards. Tests are carried out for a variety of reasons. Tensile test results are used to choose materials for engineering purposes. To assure the quality of rebar, tensile characteristics are commonly specified in material specifications. Such characteristics are frequently tested during the development of novel materials and processes in order to compare different materials and processes. Finally, tensile characteristics are frequently utilized to forecast how a material will behave under loads other than uniaxial tension.

The strength of a material is quite often the most important consideration. The stress required to cause appreciable plastic deformation or the maximum stress that the material can withstand are two ways to measure the strength of interest. These strength measures are used in engineering design with appropriate caution (in the form of safety factors). The material's ductility, which is a measure of how much it can be deformed before fracturing, is also of interest. Ductility is rarely directly incorporated into the design; rather, it is included in material specifications to ensure quality and toughness. Low tensile ductility is frequently accompanied by low resistance to fracture under other types of loading. Elastic properties may also be of interest, but special techniques must be used to measure these properties during tensile testing, and ultrasonic techniques can provide more accurate measurements. This chapter provides a brief overview of some of the more important testing procedures. There are many different properties that are important in the design and construction of buildings and structures. These properties include:

a. Diameter and unit weight	e. Bendability
b. Yield strength	f. Surface deformation
c. Tensile strength	g. Fatigue
d. Elongation	h. Chemical composition
	i. Bond testing

Testing of the above-mentioned properties is discussed in the following sections.

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3.2 SAMPLING

Testing rebar starts with sampling. Proper sampling in accordance with code provisions is essential to maintain uniformity and reliability of the testing and quality evaluation process. The frequency of sampling requirement shall generally be established by standard code provisions as well as by the requirement of the owner of a project. Generally, it is standard practice to collect one batch of samples for every 50 tons of reinforcement supplied at site. Alternatively, one batch of samples for the construction of every floor may be collected and tested when less than 50 tons of reinforcement is used on a floor. Each batch should consist of samples of all rebar sizes (diameter), and there should be three samples for each rebar size. The collection of samples from a stored stack should be as random as possible.

Generally, 600 mm long samples are required for tension testing. Testing for unit weight can be conducted prior to tension testing. For the bend test, an additional 600 mm long sample shall be needed. If both tension and bend tests are needed, 1200 mm long samples may be collected and used in testing.

3.3 DIAMETER AND UNIT WEIGHT

Unit weight of rebar means the weight of a sample of unit length. When rebar is supplied for testing, the weight and length of each sample are recorded (Figures 3.1 and 3.2). Then the weight of a sample is divided by its length to get its unit weight. Generally, the average unit weight of three samples is taken.

Currently, plain bars are rarely used. Virtually all rebar has surface deformation, which aids in developing bonding with concrete through mechanical interlocking. In the presence of surface deformations (Figures 2.1 and 2.2) the idea of *diameter* is a little obscured. In rebar standards such as ISO 6935-2 (or BDS ISO 6935-2) or ASTM A615, a term called *nominal diameter* is used. The nominal diameter does not represent any particular measurement of rebar. Instead, it is just an indication of

FIGURE 3.1 Measurement of sample weight.

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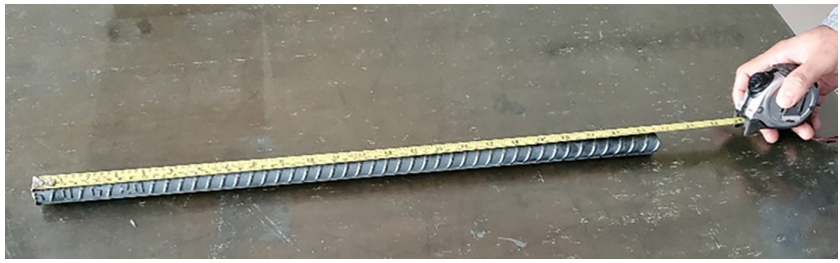

FIGURE 3.2 Measurement of sample length.

a rebar size that approximates its diameter or width. The nominal diameter is defined as the diameter of an equivalent perfectly round plain bar of the same material having the same mass or weight per unit length. Following this definition, the diameter of the rebar can be determined using Equation 2.1 as explained in Section 2.1.2.

3.4 TENSILE TESTS

Tensile properties of materials are, by far, the most important of all the different engineering properties of any material. This section provides a brief overview of some of the more important topics associated with tension testing of rebar. These include the preparation of specimens for testing, test machines, collection of data, etc.

3.4.1 Testing Machines

Rebar is typically tested in universal testing machines (UTM). These machines are generally capable of testing material samples in both tension and compression. Using proper accessories or attachments, bending and certain other tests can also be performed by these machines. These machines come in several different ranges of load capacity. Testing machines are either electromechanical or hydraulic. Generally, hydraulic machines have a higher capacity. The main difference is the method by which the load is applied. Electromechanical machines are powered by a variable-speed electric motor, a gear reduction system, and one, two or four screws that raise and lower the crosshead. This motion applies tension or compression to the specimen. The crosshead speed can be adjusted by adjusting the motor speed. To precisely control the speed of the crosshead, a microprocessor-based closed-loop servo system can be implemented. In hydraulic machines, the load is applied hydraulically using a piston-cylinder system. Hydraulic fluid is pumped into the chamber of the cylinder, which pushes up the piston. A strong platform with two or four columns and a crosshead (moving) is attached on top of the piston. The sample is gripped between a fixed crosshead and the moving crosshead. Load is applied on the sample when the moving crosshead moves up due to pressure applied on the piston. The capacity of a hydraulic system as high as 300 tons is common. A typical picture of a modern 100-ton capacity hydraulic universal testing machine is shown in Figure 3.3.

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3.4.2 Rebar Specimen for Tension Testing

According to international standards such as ASTM A615 or ISO 6935-2, full-size rebar shall be tested for strength. The length of the specimen shall be such that it can be easily mounted on the machine, and at the same time, sufficient gage length is available between the jaws or grips of the machine for measurement of strain or elongation. Generally, a length of 600 mm is sufficient for conducting a tension test of rebar. A typical tension test produces three results – yield load, tensile load (ultimate load) and elongation data. The load data are generally displayed by the machine on the screen or dial. Most of the machines also have the provision of automatic recording of data for further processing.

3.4.3 Elongation Measurement

The test sample must be appropriately prepared so that elongation data can be read after testing. Elongations are measured on the basis of specific gage length. This gage length varies among different standards. Table 3.1 shows the gage length to be used for rebar testing according to different standards.

TABLE 3.1
Gage length of rebar in different standards

Standard	Gage length
ASTM (A615, A706, etc.)	200 mm
ISO 6935-2	$5.65\sqrt{A} \approx 5d_b$, where A is the cross-sectional area of rebar and d_b is the diameter.

Before testing, gage marks are placed on the specimen along its length at certain intervals. Such marking is done using a felt-tipped ink pen or a pointed steel punch. After testing, the sample is elongated. This causes an increase in distance between the gage marks. By comparing the increased distance to the original distance between two particular gage marks, the amount of elongation can be determined. It will be an ideal case if the sample breaks at a location midway between two gage marks. However, in reality, this does not always happen. To facilitate a reliable measurement of elongation, gage marks are placed on the specimen at closer intervals than the required gage length. For example, gage marks are generally placed at 50 mm intervals for testing according to ASTM standards. After the sample is broken in tension, four consecutive gage mark intervals (i.e. $4 \times 50 = 200$ mm before testing) are chosen in such a manner that the broken location falls approximately midway between this group of four intervals. Thereafter the elongated length is measured from this group of four intervals, and the amount of elongation is determined (Figure 3.4).

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FIGURE 3.3 A modern 100-ton capacity universal testing machine.

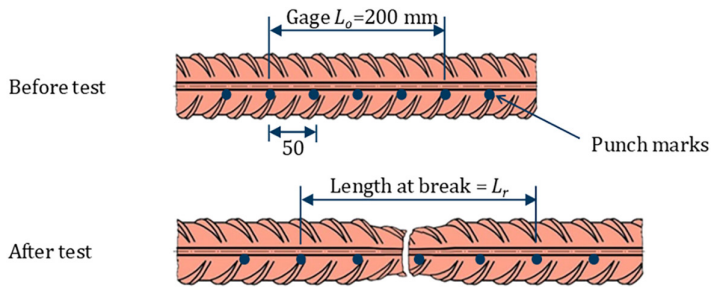

FIGURE 3.4 Marking of a specimen for elongation measurement.

3.5 BEND TESTING

Bendability of rebar is an important characteristic. This is an indirect measure of ductility. Bend testing of rebar is performed by bending a piece of rebar around a pin with a specific diameter. The bend-test specimen must withstand being bent around a pin without cracking on the bent portion's outside radius. The requirements for

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sizes of pins and the degree of bending are prescribed in Tables A.9 through A.12 in the Appendix. When the material is furnished in coils, the test specimen shall be straightened prior to placing it in the bend tester.

Bend tests must be performed on specimens of sufficient length to ensure free bending and with an apparatus that provides continuous and uniform force application throughout the bending operation. The bending operation shall also ensure unrestricted movement of the specimen at points of contact with the apparatus and bending around a pin free to rotate. There shall be a close wrapping of the specimen around the pin during the bending operation. Photographs of a typical bend testing machine are shown in Figure 3.5. The preferred method of bending is to bend the rebar around a mandrel of a specified diameter. In this process, one end of the bar is held fixed while the other end of sufficient length is wrapped around the mandrel, as shown in Figure 2.7.

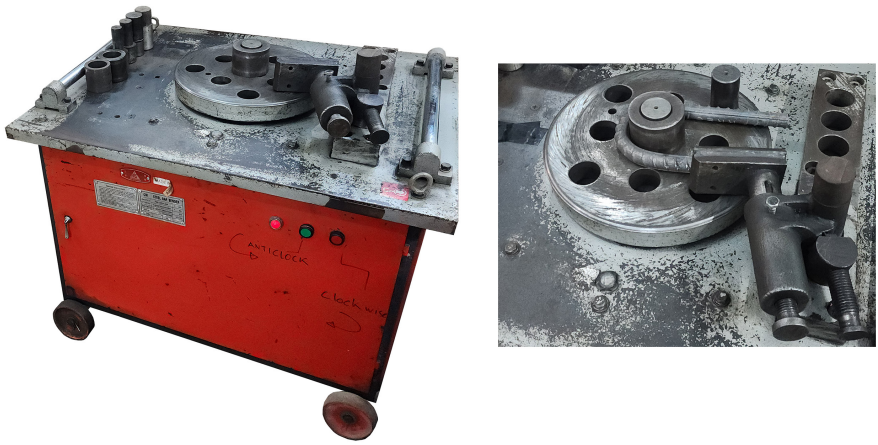

FIGURE 3.5 A typical rebar bending machine.

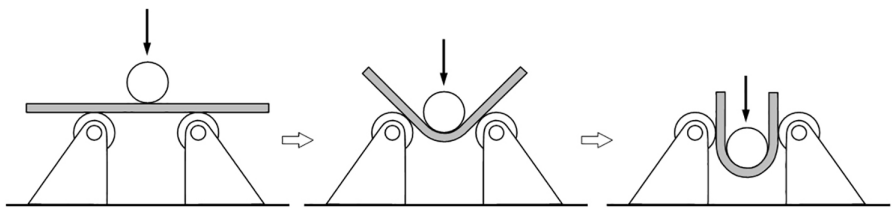

FIGURE 3.6 Different stages of the alternate bending method.

It is generally permissible to use other methods of bend testing, such as placing a specimen across two round bearings free to rotate and applying the bending force with a fixed rounded-tip mandrel conforming to the specified bend radius, allowing the bar to pass through with sufficient clearance. A typical arrangement is shown in

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Figure 3.6. Such an arrangement is generally used with universal testing machines. When failures occur under this methods of bend testing, retests are generally recommended following the procedure described in the previous paragraph.

A properly ductile MS rebar should bend to an appropriate angle without showing any sign of distress or cracking on the outer periphery of the bent bar. After bending, the samples shall be visually inspected for occurrences of cracks on the tension surface of the bend specimen. If the tension surface does not show any unusual texture or crack, then the bend test may be considered satisfactory.

3.6 MEASUREMENT OF SURFACE DEFORMATION

Requirements for surface deformation are discussed in Chapter 2. The key deformation parameters are (see Figure 2.2):

a. Transverse rib height.
b. Transverse rib spacing.
c. Transverse rib inclination.
d. Transverse rib gap.
e. Height of longitudinal rib.

The above parameters are measured manually for samples of reinforcement using simple laboratory instruments like slide calipers, measuring tape, protractors, etc. In the test report, the above measurements are presented for three samples of a particular bar size and their average value.

3.7 STRESS VS. STRAIN DIAGRAM

While designing reinforced concrete structures, an engineer generally does not need the stress vs. strain diagram (Figures 2.4, 2.5 and 2.8). However, in order to determine parameters like elongation at maximum force (ε_{emf} in Figure 2.4) and Young's modulus, determination of the stress vs. strain diagram is essential. Testing is generally done in a universal testing machine following the procedure of testing tensile properties of rebar. In order to determine the strain, a device called an *extensometer* is attached to the specimen. The extensometer can be either mechanical, having a dial gauge, or an electronic device such as a linearly variable displacement transducer (LVDT) capable of producing electronic signals depending on the position of a slider. Typical pictures of extensometers fitted on rebar samples are shown in Figure 3.7.

When a mechanical extensometer is used, readings are to be taken manually at certain intervals of loading. Thereafter, the stress vs. strain diagram is manually drawn based on the data read from UTM. Once the diagram is drawn, the required parameters can easily be obtained from the graph. Modern UTMs use electronic extensometers, which automatically record the strain data along with the loading

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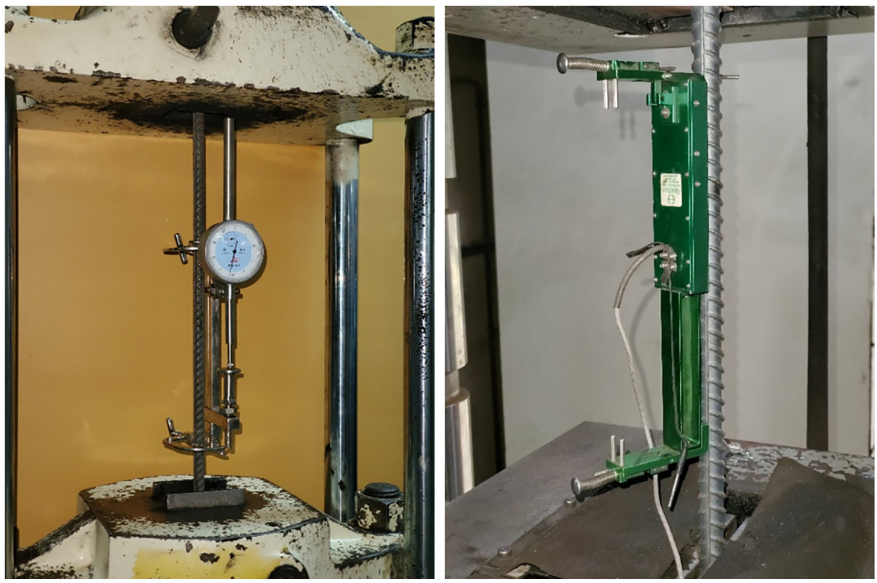
data. These modern UTMs can also automatically produce the stress vs. strain diagram and report Young's modulus, elongation at maximum force, as well as other parameters. A typical stress vs. strain diagram produced by a modern electronically controlled UTM is shown in Figure 3.8.



(a) Mechanical extensometer (b) LVDT type electronic extensometer

FIGURE 3.7 Typical extensometers for rebar testing.

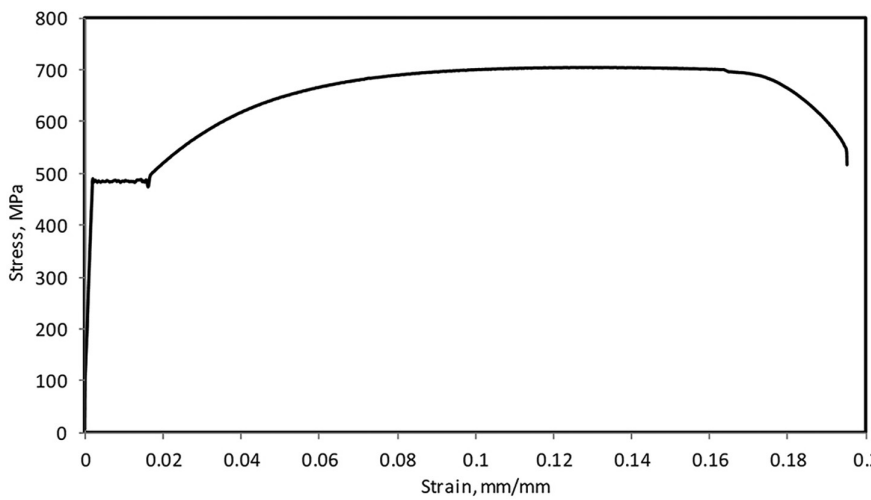


FIGURE 3.8 Typical stress vs. strain diagram of a rebar.

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3.8 FATIGUE TESTING OF REBAR

Fatigue testing of mild steel rebar samples is done by fatigue testing machines. In fatigue testing, the rebar sample is subject to cyclic tensile loading. Depending on the arrangement by which the loading is applied, fatigue testing can be classified as an (a) axial test or (b) bend test.

3.8.1 Axial Fatigue Testing of Rebar

Axial fatigue tests are performed on conventional fatigue testing machines with as-delivered specimens. The benefits of axial tests include low costs at a relatively high testing frequency (up to 150 Hz) and the ability to precisely define pre-set stresses. A significant disadvantage is related to issues with clamping bars in testing machines, where local stresses in bar clamping areas cause the bars to fail prematurely. Secondary stresses caused by non-rectilinearity of specimens or misalignment of testing machine jaws are also difficult to avoid. There have been numerous attempts to properly prepare the grip parts of specimens; such attempts were always connected with using an additional layer between a specimen and a testing machine jaw. Such attempts included the use of leather belts wrapped around the specimen grip part, low melting point metallic fillings, resins and aluminum pads, etc. However, the effectiveness of such attempts is not well documented. A photograph of a typical axial fatigue testing machine is shown in Figure 3.9.

Both ASTM and ISO standards provide guidelines for fatigue testing of metallic materials. ASTM E466 covers fatigue testing of metal specimens. This standard requires the preparation of samples in a specified geometry. On the other hand, fatigue testing of rebar generally requires testing of unmodified samples as supplied. Therefore, fatigue testing of rebar in accordance with ASTM E466 may not be fully appropriate for concrete rebar. European standard ISO 15630-1 provides a detailed guideline for fatigue testing of concrete rebar. A summary of the ISO 15630-1 procedure is described in the following paragraphs.

According to ISO 15630-1 specification, the axial load fatigue test consists of subjecting the test piece to an axial tensile force that varies cyclically in the elastic range according to a sinusoidal wave-form of constant frequency f (see Figure 3.10). The test is repeated until the test piece ruptures or until the number of load cycles specified in the relevant product standard is reached without rupture. For the fatigue test, the straightness of the specimen is crucial. A production straightening machine may be utilized to achieve satisfactory straightness. The surface of the free length between the grips must not be treated or marked in any way. The minimum free length is 140 mm or $14d_b$, whichever is greater.

The fatigue test equipment has to be properly calibrated. The test piece shall be gripped in the test equipment in such a way that force is transmitted axially, and the test piece is free of any bending moment along its length. The upper force (F_{up}) and the force range (F_r) shall be as given in the relevant product standard. The test shall be carried out under conditions of stable upper force (F_{up}), force range (F_r) and frequency (f). Throughout the test, the cyclic loading should not be interrupted.

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FIGURE 3.9 A typical fatigue testing machine.

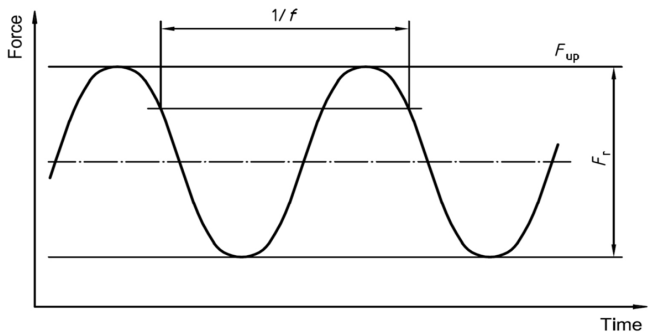

FIGURE 3.10 Load cycle diagram.

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However, according to ISO 15630-1, it is permissible to continue a test which is accidentally interrupted. Any interruption shall be reported. The frequency of load cycles shall be stable during the test and also during the test series. It shall be between 1 Hz and 200 Hz. The temperature of the test piece shall be kept below 40°C throughout the test. The temperature of the testing laboratory shall be between 10°C and 35°C, unless otherwise specified. For tests carried out under controlled conditions, the temperature of the testing laboratory shall be (23 ± 5)°C. Upon failure of the test piece before reaching the specified number of cycles or upon completion of the specified number of cycles without failure, the test shall be terminated. If the failure occurs in the grips or within a distance of two times the bar diameter of the grips or initiates at an exceptional feature of the test piece, the test may be considered invalid.

3.8.2 Bend Fatigue Testing of Rebar

Bend fatigue tests are typically carried out on concrete beams with a single reinforcing bar. The most significant benefit of these tests is that they take into account the real mutual effect of steel and concrete during testing; however, a disadvantage is the low testing frequency of 3 Hz, which is due to significant beam bend ranges and the need to avoid friction-induced beam heating. Active life tests of 10^7 cycles would take about six weeks in this case, significantly increasing the cost. Another disadvantage is that it is necessary to make assumptions about the load carrying capacity of the concrete and the accuracy with which the reinforcement is placed in order to calculate the stresses. Other drawbacks include having to make certain assumptions about a force acting on concrete and having to precisely place a bar in concrete. Bend fatigue tests are carried out in accordance with DIN 488-2:2009 on beams subjected to classic bending as well as three-point or four-point bending. A load in a beam is usually applied in a sinusoidal pattern with a constant force amplitude. Comparative tests revealed that bent beams have a slightly higher fatigue strength than specimens loaded axially.

Pfister and Hognestad (1964) conducted a research on three-point bending to simulate conditions at the center of short spans, four-point bending to simulate long spans, and bent beams as used in the German DIN specification. They found that the bent beam arrangement has consistently lower fatigue strengths than the four-point bending of a straight beam. In order to better simulate practice, tests were also conducted on beams with up to four reinforcement bars. Some researchers use a thin strip of the material cast into the surface at the center of the concrete beam to initiate cracking in four-point bending. Although Soretz (1974) found it unsatisfactory for his tests because the lives were longer than those obtained with natural cracking, it appears to be an efficient way to control the position of the fracture.

Hydraulic jacks with spreader beams are used to load the majority of the bending tests for four-point bending. Load cycles are typically sinusoidal and controlled to maintain a constant load amplitude. Hydraulic equipment, on the other hand, is costly in terms of both capital and operating costs. As a result of prolonged testing, occupancy is demanded at frequencies less than 3 Hz, and only a few tests have been run for durations exceeding 10^7 cycles. A simple mechanical test rig with a

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small electric motor and cranks has been designed to solve this problem. Maximum loads change with specimen stiffness because the cycles are deflection controlled. In practice, cracking appears in the concrete early in the testing process and quickly stabilizes, resulting in a test with a constant load amplitude. The cyclic stresses experienced by the reinforcement are constantly maintained throughout the testing by manually measuring loads and adjusting deflections. These machines take up a small amount of floor space and are best used in batches of ten or less. Long-term tests lasting up to a year can be conducted easily with such arrangements. The system has been successfully modified to fatigue beams in brine baths. Bending tests in a liquid environment can be challenging because it has been discovered that the overall stiffness of a reinforced beam increases during cycling, making it impossible to calculate stresses in the reinforcement. This is thought to be due to debris wedging cracks open, a mechanism that does not occur in air tests.

Comparative tests have shown that bending can result in slightly longer lives than axial loading, but the differences are not significant. For example, Soretz (1965) discovered that the endurances of cold-twisted bars (now defunct) tested in bending were about 10% better than axial tests. Data for steel with a yield strength of 410 MPa (Walker *et al*. 1975; Bannister 1975; TRRL 1978) show that there is no significant difference in fatigue strength within the scatter. Because the highest stresses are limited to the parts of the bar furthest from the neutral axis and in the vicinity of cracks in the concrete, bending tests should have longer endurances. It's unlikely that these locations will coincide with the bars' worst flaws. Axial tests, on the other hand, apply uniform nominal stress to the bar, allowing the fracture to start from the worst defect.

3.9 CHEMICAL COMPOSITION TEST

Various methods, such as direct chemical analysis by titration, spectrometric method and others, can be used to determine the chemical composition of steel products. For chemical composition analysis of rebar, ASTM or ISO standards typically prescribe spectrometric methods. The wavelength and intensity of electromagnetic radiation are measured in the spectrometric method of chemical analysis. Its main application is in determining the arrangement of atoms and electrons in chemical compound molecules based on the energy absorbed during changes in the structure or motion of the molecules. Two methods are usually implied in their restricted and more common usage: (1) ultraviolet (non-visible) and visible emission spectroscopy and (2) ultraviolet, visible and infrared absorption spectroscopy. Electrical discharges (arcs, sparks) or flames are used to excite atoms to energy levels higher than their lowest normal levels (ground states) in emission spectroscopy. When excited atoms return to lower energy states, they emit light with specific frequencies, which can be used to identify the elemental composition of an unknown substance. Diffraction or refraction (deflection of the path of light by a grating or a prism) separates these characteristic frequencies into an ordered sequence (spectrum) for observation in a spectroscope (visual), spectrograph (photographic) or spectrometer (photoelectric). There are four interdependent steps in the process: (1) vaporization of the sample, (2) electronic

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excitation of its atoms or ions, (3) dispersion of the emitted or absorbed radiation into its component frequencies and (4) measurement of the intensity of the radiation, usually at its most intense wavelengths. Emission spectrochemical analysis is commonly used to determine the qualitative and quantitative properties of metallic elements, but it is not limited to that. The most sensitive of all analytical methods: only a few milligrams of a solid sample are usually required to detect metallic elements present in concentrations of a few parts per million or less. Furthermore, the method can detect multiple atomic species at the same time, eliminating the need for chemical separations. A spectrometer is typically used to perform spectrometric chemical analysis on rebar. A picture of a typical spectrometer is shown in Figure 3.11.

FIGURE 3.11 A typical optical emission spectrometer.

3.10 BOND TESTING

The bond between reinforcing bars and concrete has long been recognized as critical to the performance of reinforced concrete structures. The ACI 408 committee makes recommendations for determining rebar bond characteristics in concrete. The bond between reinforcing bars and concrete has been studied using a variety of test specimen configurations. Figure 3.12 depicts the four most common configurations. The nature of the bond response is affected by the details of the specimen, as well as the measured bond strength.

Because of its ease of fabrication and simplicity of test procedure, the pullout specimen of Figure 3.12 (a) is widely used. When the bar is in tension, transverse reinforcement is often used to prevent the bar from splitting. Because the stress fields within the specimen may not match the cases in actual construction, this specimen is the least realistic of the four shown in Figure 3.12. ACI Committee 408 does not recommend that this arrangement of pullout test results be used as the sole basis for determining development length.

Figure 3.12 (b) through (d) shows specimens that provide more realistic measures of bond strength in actual structures. The modified cantilever beam, or beam-end

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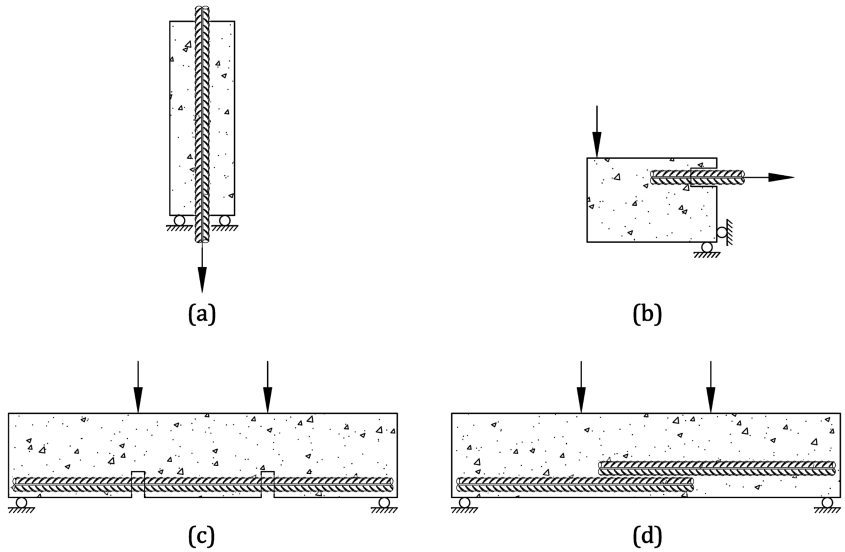

FIGURE 3.12 Arrangements for bond testing of rebar (ACI 408R-03).

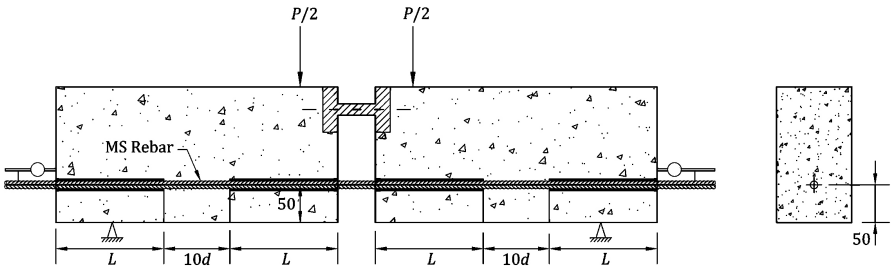

FIGURE 3.13 Beam test for determining bond characteristics of rebar by BS 4449.

specimen, shown in Figure 3.12 (b), is a relatively simple test that generally duplicates the stress state found in reinforced concrete members; the reinforcing steel and surrounding concrete are both placed in tension at the same time. The ASTM A944 standard covers bond strength testing using the arrangement shown in Figure 3.12 (b). The beam anchorage and splice specimens in Figure 3.12 (c) and (d), respectively, are larger-scale specimens used to measure development and splice strength in full-size members. Figure 3.12 (c) shows an anchorage specimen that simulates a member with a flexural crack and a known bonded length. The splice specimen in Figure 3.12 (d), which is normally fabricated with the splice in a constant moment region, is easier to fabricate and produces bond strengths that are comparable to those obtained with the anchorage specimen.

British Standard BS 4449 also covers the determination of rebar bond characteristics. A beam specimen, as shown in Figure 3.13, is prepared and tested under

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two-point loading in this method. The beam test is used to determine the bond strength of reinforcing steel and to compare reinforcing bars and de-coiled products with similar diameters but having different surface configurations. Reinforcing steel with a diameter of 32 mm or less can be tested using this method. A specially detailed test beam is loaded using simple four-point flexure until the bond fails completely in both half-beams or the reinforcing steel itself fails. The slip of the two ends of the reinforcing steel is measured during loading. The test beam is made up of two parallelepiped reinforced concrete blocks that are connected at the bottom by the reinforcing steel that will be tested and at the top by a steel hinge. Figure 3.14 shows a typical rebar axial force vs. slip curve obtained from a bond test.

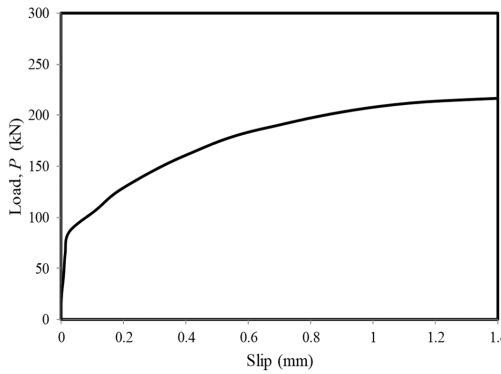

FIGURE 3.14 Typical axial load vs. slip curve obtained from the bond test.

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QUESTIONS

3-1. Which properties of rebar are important in the design of concrete structures? Discuss.
3-2. Why is sampling important for rebar testing? Describe the procedure for sampling and testing rebar.
3-3. What do you understand by nominal diameter? How is it measured?
3-4. What properties of rebar are determined through tensile testing? Discuss.
3-5. What is the bend test of rebar? What engineering characteristic property is ensured by bend testing?
3-6. Explain the process of drawing stress vs. strain diagrams using mechanical and electronic extensometers.
3-7. How is axial fatigue testing of rebar performed? Describe the relevant ASTM and ISO standards for fatigue testing. Which standard is preferred for rebar and why?
3-8. What is bend fatigue testing of rebar? Describe the four-point bend fatigue test. Which test is preferable – axial or bend fatigue?
3-9. Which method is prescribed for testing chemical composition according to ASTM and ISO standards? Describe.
3-10. With neat sketches, briefly describe the bond test procedures according to ACI and BS standards.

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4 Characteristics of High Strength Rebar Produced around the World

4.1 GENERAL

In the United States, reinforcement with a yield strength of up to 120 ksi is now available (ATC 115). Building components resisting earthquake effects in Japan are currently reinforced with 100 ksi yield strength reinforcement. ASTM specifications A615, *Standard Specification for Deformed and Plain Carbon-Steel Bars for Concrete Reinforcement*, and A706, *Standard Specification for Low-Alloy Steel Deformed and Plain Bars for Concrete Reinforcement* were the first to include Grade 80 reinforcement. In the United States, Grade 80 reinforcing bars are manufactured, but they are typically made for specific project requirements rather than for general use. However, as it is used more frequently, its availability is expected to rise. Grade 100 and Grade 120 reinforcement are covered by ASTM A1035, *Standard Specification for Deformed and Plain, Low-Carbon, Chromium, Steel Bars for Concrete Reinforcement*. ACI ITG-6R-10, *Design Guide for the Use of ASTM A1035/A1035M Grade 100 Steel Bars in Structural Concrete*, recommends using the higher yield strength to increase the flexural and axial strength of members. SAS 670 is a reinforcement material manufactured in Germany for use in columns. The International Code Council (ICC) evaluation report provides an assessment of this material for use in the United States (ICC 2011). In New York City, this reinforcement was used in high-rise construction (NIST 2014).

Other countries have several types of high-strength reinforcing bars that are not widely available in the United States. Japan's New RC Project investigated high-strength reinforcement in the early 1990s (Aoyama 2001). In 1995, Japan introduced high-strength reinforcement specifications, coinciding with a surge in the number of reinforced concrete high-rise buildings being built. In Japan, approximately 200 high-rise reinforced concrete buildings (defined as 60 meters (197 feet) or higher) were built between 1995 and 2002 (Sugano 2008). Currently, high-strength reinforcing bar types developed and approved in Japan in accordance with JIS G 3112:2010 are (a) SD590A and SD590B, having a yield strength of 590 MPa min, (b) SD685A, SD685B and SD685R having a yield strength of 685 MPa min and (c) SD785R with a yield strength of 785 MPa min. Aoyama (2001) reported that the use of higher grade rebar such as USD980 and USD1275 were also proposed and were experimentally

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used. The Ministry of Construction in Japan has approved these reinforcement types as part of the New RC Construction Standard (Aoyama 2001). However, the incorporation of these high-strength materials into JIS standards is still pending.

Grade 500 (72.5 ksi) reinforcement is produced in New Zealand, Australia and China. Grade 500E, which is specifically designed for use in members designed to resist earthquake forces, and Grade 500N, which has less stringent ductility requirements than Grade 500E, are both included in the Australian/New Zealand specification AS/NZS 4671 (AS/NZS 2001). Grade 500N reinforcement is commonly used in Australia due to its low seismicity. Meanwhile, New Zealand uses Grade 500E reinforcement. HRB500 and HRBF500 reinforcing bars are widely available in China and are included in Chinese building code standards. A comparison was made between the HRB500 reinforcement available in China and the Grade 500N reinforcement available in Australia and New Zealand by Li *et al.* (2010). It was found that by adding the trace elements V, Nb and Ti into the steels, the seismic ability of Chinese steels has been improved. Because of the higher percentage of carbon, the tensile strength of Chinese reinforcing steel is generally higher than that of Australia's normal ductility (N) reinforcing steel. The weldability of Australian steel was found to be higher than Chinese steel.

4.2 ASTM A615 PRODUCTS

Rebar standard ASTM A615 was introduced in 1968 with three grades – 40, 60 and 75. Later in 1972, Grade 75 was withdrawn. In the year 1987, Grade 75 was reintroduced in A615. Grade 80 was incorporated in 2009. Later in 2015, Grade 100 was introduced in A615. Deformed and plain carbon steel bars in cut lengths and coils for concrete reinforcement are covered in this specification.

4.2.1 Tensile Requirements

The tensile strength requirement of ASTM A615 for grades 60, 75, 80, and 100 are shown in Table A.5 in the Appendix. One of the following methods must be used to calculate the yield point or yield strength. When the steel tested has a sharp-kneed or well-defined yield point, as shown in Figure 2.4, which is generally characterized by a well-defined yield plateau, the yield point shall be determined by the drop or halt of the gauge of the tensile testing machine. When the steel being tested lacks a clearly defined yield point, the yield strength must be calculated using the offset method (0.2% offset), as shown in Figure 2.5 and as described in ASTM Test Methods and Definitions A370. The elongation requirements of A615 bars are shown in Table A.5 in the Appendix. Elongation shall be measured in 200 mm gage length for all bar sizes. ASTM A615 reinforcement is the most commonly used reinforcement in the United States. Although it is not specifically intended for use in members resisting earthquake effects, ACI 318-11 allows its use in special moment frames and boundary elements of special structural walls if the following strength requirements are met (these are the same strength requirements for ASTM A706 reinforcement):

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a. Mill tests show that the actual yield strength does not exceed specified yield strength, f_y, by more than 18,000 psi.
b. The actual tensile strength must be equal to or greater than 1.25 times the actual yield strength.

4.2.2 Bend Requirements

Bend requirements for ASTM A615 rebar are shown in Table A.9 in the Appendix. It is to be noted that the diameter of the pin may be smaller than the same prescribed by ACI 318. On the other hand, the bend requirement may be less stringent than those of ASTM A706. The bend-test specimen shall be considered satisfactory when there shall be no visible cracks on the outside radius of the bent portion after being bent around a pin. The test shall be done by bending rebar around a pin of specified diameter, as shown in Figure 2.7. An alternate method of bending, as shown in Figure 3.6 may be followed. However, if unsatisfactory results are obtained in such an alternate method, a retest shall be made following the preferred method in Figure 2.7.

4.2.3 Surface Deformation Requirements

The surface deformation requirements of the A615 rebar are explained in detail in Section 2.1.3 of this book. The summarized requirements are given in Table A.3 in the Appendix.

4.3 ASTM A706 PRODUCTS

Rebar standard ASTM A706 was first introduced in 1974 with improved weldability, ductility and strength characteristics. At that time, only Grade 60 rebar was included. Later in 2009, Grade 80 rebar was in A706 specification.

4.3.1 Tensile Characteristics

A706 specification puts restrictions on the measurement of yield strength when the stress-strain curve does not show any sharp kneed and well-defined yield plateau. The specification states that, in the absence of a well-defined yield plateau, the yield shall be determined by the 0.2% parallel offset method, and at the same time the stress corresponding to 0.0035 strain shall not be less than 60 ksi or 80 ksi for Grade 60 or Grade 80 rebar respectively. The strength requirement at 0.0035 strain was also included in ACI 318-11 and earlier versions. The 0.35% strain limit was felt necessary to ensure that the assumption of an elastoplastic stress-strain curve will not lead to unconservative values of the member strength calculated using ACI 318 specification. However, the requirement of a 0.35% strain limit was dropped from the recent ACI 318-14 because of the fact that the definition and determination of yield in accordance with ASTM A615-14 or A706-09 is now more streamlined and well-defined. However, BNBC 2020 still maintains the 0.35% strain requirement. Tensile strength properties of ASTM A706 Grade 60 and Grade 80

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reinforcement are provided in Table A.6 in the Appendix. Specification of ASTM A706 requires that the tensile strength of the reinforcement be at least 1.25 times the actual yield strength. This requirement, which is the same for Grade 60 and Grade 80 reinforcement, is considered important to obtain an adequate spread of plasticity in regions where yielding is expected, i.e., plastic hinges. Examples of stress-strain curves for ASTM A706 Grade 60 and Grade 80 reinforcements are shown in Figure 4.1.

ASTM A706 Grade 80 reinforcement was the first high-strength reinforcement that allowed to be used within the United States in designs to resist earthquake effects. The properties of this reinforcement are similar to, but not exactly the same, as ASTM A706 Grade 60 reinforcement.

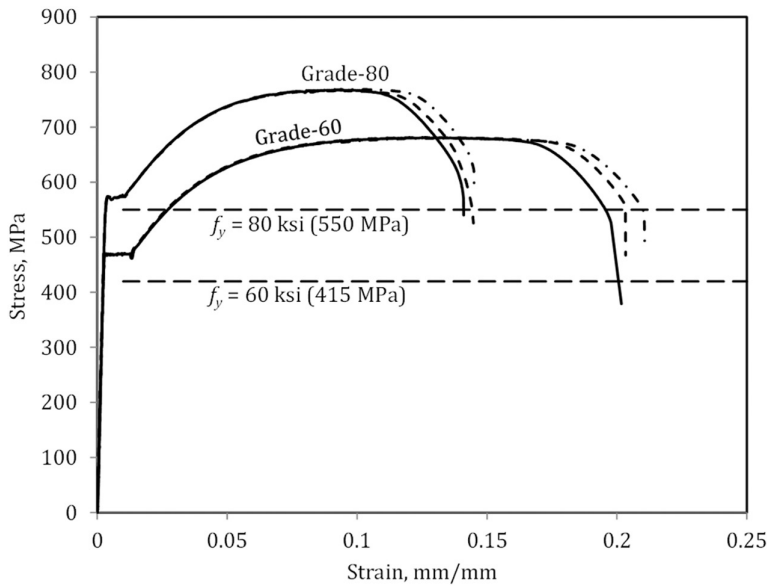

FIGURE 4.1 Example of stress-strain curves for ASTM A706 Grade 60 and Grade 80 reinforcement.

4.3.2 Elongation Requirement

Elongation is an important measure of the suitability of reinforcement for use in members resisting earthquake effects. Although total elongation requirements for Grade 80 reinforcement are not the same as those of Grade 60 reinforcement for all bar sizes, it is the same for bar sizes 7, 8, 9, 10, 11, 14 and 18. These sizes are most likely to be specified for primary longitudinal reinforcement; therefore, they are most likely to yield in members resisting earthquake effects. The elongation requirement for rebar in A706 specification is listed in Table A.6 in the Appendix.

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4.3.3 Bend Requirement

An indication of ductility is the ability to bend reinforcement about a set diameter. Table 4.1 includes the ASTM A706 bend test requirements for Grade 60 and Grade 80 reinforcement given as a multiple of the bar diameter, d_b, as well as the required bend diameter for stirrups and hoops and for standard hooks specified by ACI 318. This table shows that the ASTM A706 bend test requirements are less stringent for Grade 80 reinforcement than for Grade 60, even though both grades of bars are bent to the same diameter for stirrups and hoops and for standard hooks. It is important to understand that the specified bend diameters of ASTM A706 serve the purpose of quality control in manufacturing, whereas the bend diameters specified by ACI 318 are for purposes of reinforcement detailing. The ACI 318 diameters are purposely established at larger values than those of ASTM A706 in an attempt to minimize the fracturing of reinforcement during bending for fabrication. It is to be noted that the bend test shall be performed with 180° bends for rebar of all diameters in A706 specification. In A615 specification 90° bend is allowed for large diameter bars. Therefore, the bend requirement of A706 rebar is more stringent than A615 specification.

TABLE 4.1

Comparison of A706 Grade 60 and Grade 80 bend test requirements to ACI 318-19 stirrup/hoop and standard hook detailing requirements (ACI 2019)

	A706 Grade 60 180° bend test	A706 Grade 80 180° bend test	ACI 318-19 stirrup/hoop bend	ACI 318-19 standard hook bend
Bar size	**Inside diameter of bend**			
10 mm through 16 mm	$3d_b$	$3.5d_b$	$4d_b$	$6d_b$
19 mm through 25 mm	$4d_b$	$5d_b$	$6d_b$	$6d_b$
29 mm through 36 mm	$6d_b$	$7d_b$	–	$8d_b$
43 mm through 57 mm	$8d_b$	$9d_b$	–	$10d_b$

d_b = rebar dia.

4.3.4 Surface Deformation Requirement

All ASTM A706 reinforcement grades must have a radius (a in Figure 2.2) at the base of each deformation that is at least 1.5 times the height of the deformation (see the rounded transition in Figure 2.2). All deformations, including transverse lugs, longitudinal ribs, grade ribs, grade marks and intersections between deformations are subject to this requirement. Newly machined rolls used to make reinforcing bars will be used to measure conformance rather than bar samples. Other surface deformation requirements are the same as those of ASTM A615 rebar (see Table A.3 in the Appendix).

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4.4 ASTM A1035 HIGH-STRENGTH REBAR

ASTM A1035 is a low-carbon chromium steel having yield strengths of 100 ksi and 120 ksi designated as Grade 100 and Grade 120, respectively in ASTM or ACI provisions. This rebar is also known as MMFX or MMFX2 steel, which stands for Martensitic Microcomposite Formable Steel. With the introduction of higher-strength steel reinforcing bars with a specified minimum yield strength of 100 ksi, the designers might be able to reduce the total cross-sectional area of required reinforcement. The reduced area of reinforcement could result in fewer bars and reduce reinforcement congestion often encountered in mat foundations, shear walls, beam-column joints and many precast concrete elements. The reduction in reinforcement congestion facilitates concrete placement and consolidation. In addition to having high strength, the steel also has very good corrosion resistance properties.

4.4.1 Tensile Characteristics

For deformed bars used as confinement reinforcement (ties or spirals) in compression members, ACI 318 permits the use of specified minimum yield strength of up to 100 ksi (690 MPa). Understanding the differences in tensile behavior of higher strength steels is essential for the safe and serviceable design of concrete members. Tensile properties are necessary in the probabilistic studies that establish the strength reduction factors used in reinforced concrete design. The ASTM A1035 bar Grade 100 (690 MPa) (Figure 4.2) exhibits a stress-strain relationship without a well-defined yield plateau. Actual yield strength, determined by the 0.2% offset method, typically exceeds 115 ksi (790 MPa) for Grade 100 bars. The tensile strength typically exceeds 155 ksi (1070 MPa) for Grade 100 bars. The corresponding strain at the peak of the stress-strain curve ranges from 0.04 to 0.06. A typical stress-strain diagram of Grade

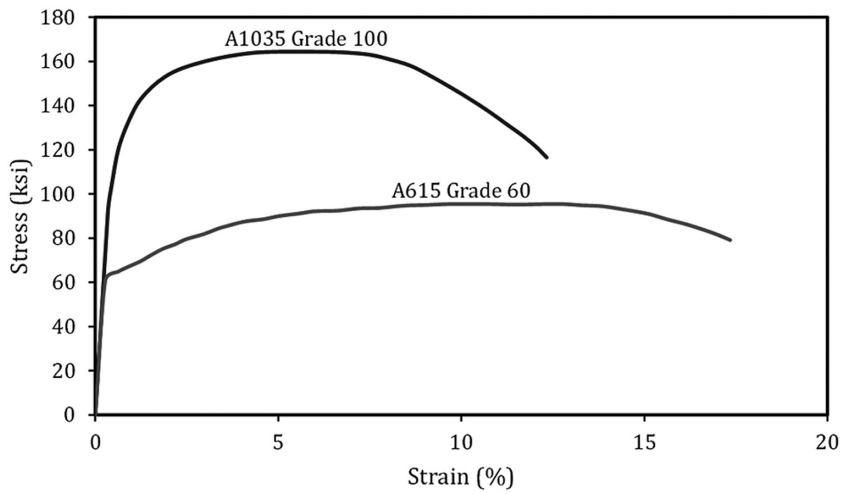

FIGURE 4.2 Typical stress-strain diagram of Grade 100 rebar (NIST 2014).

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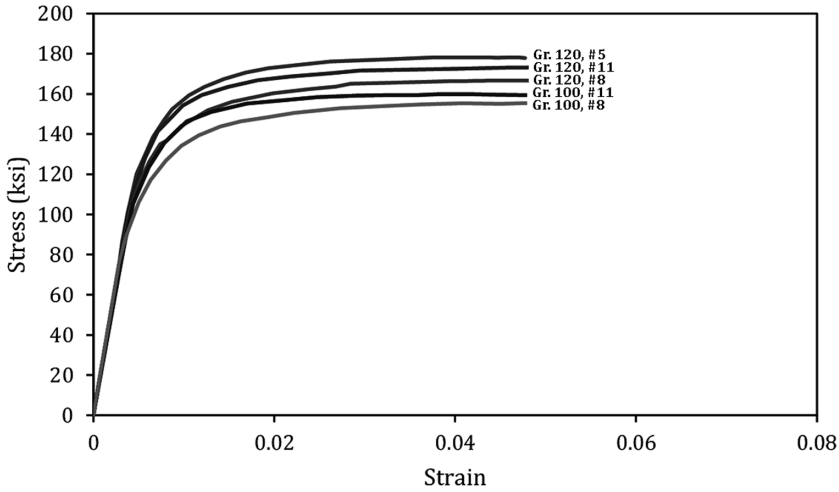

FIGURE 4.3 Stress-strain curve for ASTM A1035 Grade 100 and Grade 120 reinforcement (NIST 2014).

100 rebar in comparison with Grade 60 rebar is shown in Figure 4.2. Examples of stress-strain curves are shown in Figure 4.3 (NIST 2014). The shape of the stress-strain curve does not include a yield plateau, and the total elongation is less than that of ASTM A706 Grade 60 and Grade 80 reinforcement. Requirements of tensile characteristics of A1035 bars are given in Table A.7 in the Appendix.

4.4.2 Chemical Composition and Weldability

ASTM A1035 bars are furnished to three different chemical compositions, designated as Alloy Type CL, CM and CS. Chemical compositions of these types are shown in Table 4.2. The specified chemical content of bars differs from A706 and A615 in that there is a required amount of chromium (8.0 to 10.9%). In addition, carbon content is limited to 0.15% and a maximum limit is specified for nitrogen (0.05%). Other than the chromium requirement, the chemical composition specified by ASTM A1035 is similar to ASTM A706. It is important to note that ASTM A1035 bars are not readily weldable because of metallurgical reasons. ASTM A1035 bars should not be welded. The heat of welding alters the microstructure of the steel.

TABLE 4.2
Chemical composition of ASTM A1035 bars

Alloy type	Carbon	Chromium	Manganese	Nitrogen	Phosphorus	Sulfur	Silicon
1035 CL	0.3	2.0–3.9	1.5	0.05	0.035	0.045	0.5
1035 CM	0.2	4.0–7.9	1.5	0.05	0.035	0.045	0.5
1035 CS	0.15	8.0–10.9	1.5	0.05	0.035	0.045	0.5

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Welding of ASTM A1035 bars should be approached with caution because no specific provisions have been included in the ASTM A1035 to enhance the weldability of the bars.

4.4.3 Tensile Requirement

To meet the requirements for tensile properties, the test specimens representing the material must meet the requirements of Table 4.3, which is a summary of Table A.7 in the Appendix. ASTM A370 *Standard Test Methods and Definitions for Mechanical Testing of Steel Products* specifies that the yield strength shall be determined using the offset method (0.2% offset). Coils of material must be unwound and the specimen straightened before it is placed in the tensile testing machine. This specification does not allow for the use of post-fabricated material for testing purposes. Table 4.3 specifies the percentage of elongation.

TABLE 4.3
Tensile requirement of ASTM A1035 bars

Type	A1035 CL, CM, CS	
Grade	Grade 100 [690]	Grade 120 [830]
Tensile strength, min, psi [MPa]	150 000 [1030]	150 000 [1030]
Yield strength (0.2% offset), min, psi [MPa]	100 000 [690]	120 000 [830]
Elongation in 8 in. [200 mm], min, %:		
Bar designation No.		
3 through 11 [10 through 36]	7	7
14, 18, 20 [43, 57, 64]	6	6

4.4.4 Bend Requirement

ASTM A1035 reinforcing bars can be bent into standard hooks by following the requirements of ACI 318. Note, however, that because of the strain-hardening characteristics of the material, it is usually necessary to overbend the bar to achieve the desired bent angle. As it is with other reinforcing bars, straightening of a bent bar embedded in concrete should be avoided, as it may cause the bar to break. The bend-test specimen shall be considered satisfactory when there shall be no visible cracks on the outside radius of the bent portion after being bent around a pin. The test shall be done by bending the rebar around a pin of a specified diameter as shown in Table A.11 in the Appendix. When the material is furnished in coils, the test specimen shall be straightened prior to placement in the bend tester.

4.4.5 Surface Geometry Requirement

Surface geometry or deformation requirements of ASTM A1035 bars are similar to those of ASTM A615 bars, and the requirements are shown in Table A.3.

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4.5 ISO 6935 REBAR

European rebar standard ISO 6935-2:2015 has been adopted as Bangladeshi standard BDS ISO 6935-2:2016. This standard covers rebar having yield strength ranging from 300 MPa to 600 MPa. Rebar of different grades are classified into four ductility classes – A, B, C and D. Here highest ductility is represented by Class D, which is applicable for the seismic design of reinforced concrete structures. In Bangladesh and elsewhere, ductility Class C rebar may be used in concrete structures where high seismic ductility is not essential. Ductility Classes A and B, though included in the specification, are rarely manufactured and used.

4.5.1 Tensile Requirements

Tensile requirements of various grades of ISO 6935-2 rebar are shown in Table A.8 in the Appendix. In this table, only the requirements of ductility Classes C and D are shown. In general, ductility Class C may be considered roughly equivalent to ASTM A615 specification. Class C covers yield strength of 300 MPa to 600 MPa for which the actual tensile strength must be 15% higher than the actual yield strength. The requirements of ductility Class D are similar to those of ASTM A706. However, while A706 covers only yield strengths 420 MPa and 550 MPa, ductility Class D covers yield strengths ranging from 300 MPa to 500 MPa. ISO standard prescribes two alternate methods of elongation measurements. The default method is to measure elongation at maximum force. However, elongation at break can also be adopted if the manufacturer and the purchaser agree.

4.5.2 Bend Requirements

Unlike ASTM standards, ISO standard specifies both bend and rebend requirements for rebar. Requirements for bend testing are given in Table 4.4 and also in Table A.12(a) in the Appendix. The methods of bend testing are similar to those described in Section 3.5 of Chapter 3.

TABLE 4.4
Bend requirement of ISO 6935-2 rebar

Nominal bar diameter, d_b (mm)	Mandrel diameter (max.)
≤16	$3d_b$
$16 < d_b \leq 32$	$6d_b$
$32 < d_b \leq 50$	$7d_b$

4.5.3 Rebend Requirements

In fabricating reinforcements for structural elements like beams, columns, etc., rebar is frequently bent to the desired shape. On some occasions, straightening or changing of bend angle is also necessary. To address proper quality control of rebar on such

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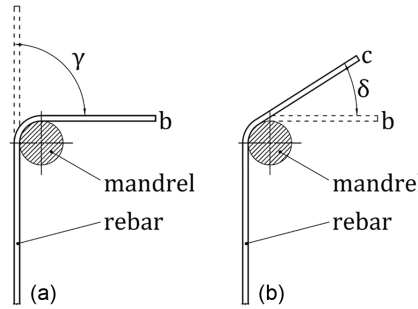

FIGURE 4.4 Illustration of a rebend test (ISO 15630).

occasions, the rebend test is specified. The procedure of rebend testing includes bending a sample to a certain angle (γ in Figure 4.4a), artificial aging and then rebending back to a certain angle (δ in Figure 4.4b). Artificial aging is generally carried out by placing the bent sample in a water bath and heating it to 100°C for about 60 to 75 minutes and then allowing it to cool in air before performing the rebend operation. The ambient temperature shall be kept within 10° to 35°C while conducting all bend and rebend operations. Generally, the absence of cracks visible to a person with normal or corrected vision is considered as evidence that the test piece has withstood the rebend test. Requirements of the rebend test are shown in Table A.12(b) in the Appendix.

4.6 ASTM A615 VS. ASTM A706 REBAR

ACI Committee 318 has approved the use of ASTM A706 Grade 80 reinforcement for special moment frames and special structural walls in the 2019 version of the code. It is likely that allowing the use of ASTM A615 Grade 75 and Grade 80 reinforcement will also be considered if the relevant requirements of ACI 318-19 are met. Therefore, this section compares the requirements and characteristics of ASTM A615 and A706 reinforcement.

Table 4.5 presents the tensile property requirements of ASTM A706 and A615 Grade 60 reinforcement. The additional ACI 318-19 requirements applicable to ASTM A615 Grade 60 reinforcement for use in seismic applications are included in the footnotes. Although the minimum strength requirements are the same when the additional ACI 318-19 requirements are applied, the minimum total elongation requirements are much different. For #7 and 8 bars, ASTM A615 reinforcement has a required minimum total elongation of 8%; for #9, 10 and 11 bars, it has a required elongation of only 7%. ASTM A706 reinforcement in contrast, has a required minimum total elongation of 12% for the same bar sizes.

Table 4.6 presents the tensile property requirements of ASTM A706 Grade 80 and A615 Grade 80 and Grade 75 reinforcement. The required minimum total elongation for A615 Grade 80 and Grade 75 #9, 10 and 11 bars is only half of what is required for A706 Grade 80 (6% vs. 12%).

In addition to differences in the required tensile properties and, notably, the total elongation, actual differences are expected in the bars that will be produced.

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TABLE 4.5
Comparison of tensile property requirements of ASTM A706 and A615 Grade 60 reinforcement

Property	A706 Grade 60	A615 Grade 60
Tensile strength, minimum	550[1] MPa	620[2] MPa
Yield strength, minimum	420 MPa	420 MPa
Yield strength, maximum	536 MPa	N.A.[3]
Elongation in 200 mm, minimum		
Bar size:		
10, 13, 16, 19	14%	9%
22, 25	12%	8%
28, 32, 36	12%	7%
43, 57	10%	7%

[1]Tensile strength shall not be less than 1.25 times the actual yield strength.
[2]ACI 318-14 requires that tensile strength shall not be less than 1.25 times the actual yield strength if used in seismic applications.
[3]ACI 318-14 requires that the maximum yield strength be 78 ksi if used in seismic applications.

TABLE 4.6
Comparison of tensile property requirements of ASTM A706 and A615 Grade 75 and Grade 80 reinforcement

Property	A706 Grade 80	A615 Grade 80	A615 Grade 75
Tensile strength[a], minimum	690 MPa	724 MPa	690 MPa
Yield strength, minimum	552 MPa	552 MPa	517 MPa
Yield strength, maximum	676 MPa	N.A.	N.A.
Elongation in 200 mm, minimum			
Bar size:			
10, 13, 16, 19	12%	7%	7%
22, 25	12%	7%	7%
29, 32, 36	12%	6%	6%
43, 57	10%	6%	6%

[a]Tensile strength shall not be less than 1.25 times the actual yield strength.

Table 4.7 provides the mean, standard deviation and coefficient of variation of total elongation for ASTM A706 Grade 60, A615 Grade 60 and A615 Grade 75 reinforcement, as reported by Bournonville *et al.* (2004), based on reinforcing bars produced in the 1990s. The table only presents data for #7 to 11 bars because these are sizes commonly used for beams of special moment frames and for vertical wall reinforcement of special structural walls. The data show that, on average, ASTM A706 Grade

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TABLE 4.7
Comparison of total elongation data of ASTM A706 Grade 60 and Grade 80, and ASTM A615 Grade 60, Grade 75, and Grade 80 reinforcement (Bournonville *et al.* 2004)

Reinforcement type	Bar size				
	22	25	29	32	36
A706 Grade 60					
Mean total elongation	15.2%	15.6%	15.8%	15.3%	14.9%
Standard deviation	2.3%	2.0%	2.0%	2.1%	1.9%
Coefficient of variation	0.1519	0.1310	0.1273	0.1339	0.1250
A706 Grade 80					
Mean total elongation	–	14.0%	–	–	14.4%
Standard deviation	–	1.6%	–	–	1.1%
Coefficient of variation	–	0.115	–	–	0.074
A615 Grade 60					
Mean total elongation	13.1%	13.2	12.6	12.5	12.3
Standard deviation	2.4	2.4	2.3	2.5	2.8
Coefficient of variation	0.1826	0.1787	0.1825	0.2035	0.2292
A615 Grade 75					
Mean total elongation	12.8%	12.2%	12.0%	11.0%	10.4%
Standard deviation	1.8%	1.7%	2.4%	1.4%	0.9%
Coefficient of variation	0.1428	0.1397	0.2018	0.1255	0.0826
A615 Grade 80					
Mean total elongation	–	11.7%	12.8%	13.3%	11.2%
Standard deviation	–	1.9%	1.7%	2.4%	1.2%
Coefficient of variation	–	0.166	0.131	0.182	0.105

60 reinforcement has higher total elongation and less variation in total elongation than A615 Grade 60 reinforcement. The same is true

for A706 Grade 80 reinforcement compared to A615 Grade 80 reinforcement. A615 Grade 75 reinforcement has lower total elongation than A615 Grade 60 and A706 Grade 60 reinforcement, with the difference being largest for the #10 and 11 bars. The typically smaller standard deviation and coefficient of variation of A615 Grade 75 and Grade 80 reinforcement relative to A615 Grade 60 reinforcement is likely the result of the smaller sample size and fewer mills providing data for the A615 Grade 75 and Grade 80 bars. The sample size for A706 Grade 80 is also smaller than that used for A615 Grade 60.

4.7 JAPANESE SD590, SD685 AND SD785 REINFORCEMENT

Current high strength steel grades as per JIS G 3112 standard are SD590, SD685 and SD785. Since the mid-1990s, Japan has used SD685 reinforcement which may be available in other Asian countries. The yield strength of this type of reinforcement is

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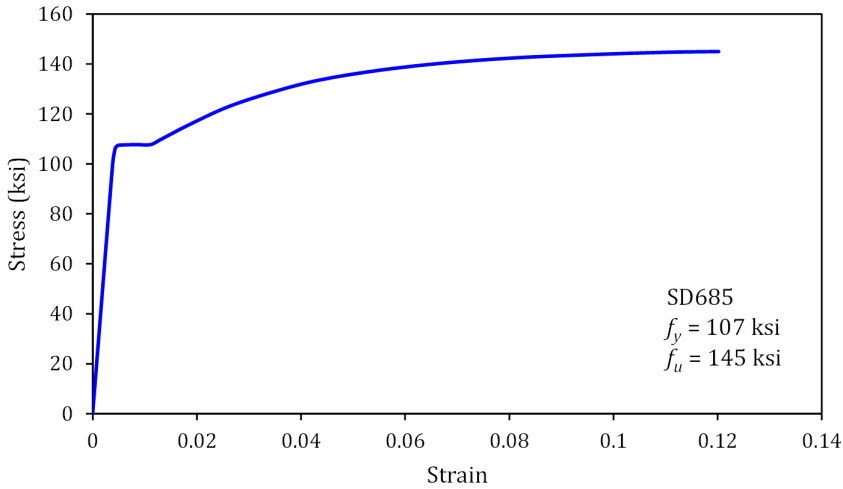

FIGURE 4.5 Stress-strain curve for SD685A reinforcement (Ousalem *et al.* 2009).

100 ksi (see Figure 4.5). Micro-alloying and adjusting rolling conditions are used to make these bars (Nishiyama 2009).

4.7.1 Tensile Strength Properties

Table 4.8 shows the tensile strength properties of high strength reinforcement as approved by Japanese standard JIS G 3112:2010. Figure 4.5 shows an example of a stress-strain curve for SD685A reinforcement. SD590A and SD590B have a nominal yield strength of 590 MPa. The upper limit of tensile strength is higher for Grade SD590B. SD590B also has a lower yield-to-tensile ratio. This grade is roughly equivalent to ASTM Grade 80 bars with about 7% higher yield strength. SD685A, SD685B and SD685R are the three types of SD685 reinforcement specified by JIS G 3112:2010. These are roughly similar to ASTM A1035 bars. SD685B has a higher minimum yield strength and a lower yield strength to tensile strength ratio than SD685A. The yield strength to tensile strength ratio must be 0.80 for SD685B, which is equivalent to ASTM A706's requirement that the reinforcement's tensile strength be at least 1.25 times the actual yield strength. SD685R has the same yield strength requirement as that of SD685A, but the yield to tensile strength ratio is not specified. To achieve a yield strength of 100 ksi, treatment after rolling or other cost-increasing changes to the manufacturing process are required. Because this treatment is not required to make USD685A, it is less expensive, and it is more commonly used wherever appropriate. SD785R has a minimum yield strength of 785 MPa and tensile strength of 924 MPa. However, the yield to tensile ratio is not specified. This bar is primarily intended for use as transverse reinforcement.

The Japanese standard has one unique specification compared to the rebar from other countries – the occurrence of a yield plateau in the stress-strain diagram is specified for the SD590A, SD590B, SD685A and SD685B grades. Yield plateau in

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TABLE 4.8
Tensile properties of SD685 rebar from Japan

	SD590A	SD590B	SD685A	SD685B	SD685R	SD785R
Yield strength, MPa	590~679	590~650	685–785	685–755	685–890	785 min
Tensile strength, MPa	695 min	738 min	806 min	857 min	806 min	924 min
Strain at the end of yield plateau	≥1.4%	≥1.4%	≥1.4%	≥1.4%	–	–
Total elongation	≥10%	≥10%	≥10%	≥10%	≥8%	≥8%
Yield to tensile strength ratio(f_y/f_{tu})	≤85%	≤80%	≤85%	≤80%[a]	–	–
90° bending pin radius	$2d_b$	$2d_b$	$2d_b$	$2d_b$	$1.5d_b$	$1.5d_b$

[a]This is equivalent to the requirement that tensile strength is 1.25 times the actual yield strength ($f_u/f_y \geq 1.25$) in ASTM A706 reinforcement.

the stress-strain diagram is a desirable characteristic as is helps to exhibit early cracks as a warning prior to failure in a distressed structure (see Section 2.4 in Chapter 2).

4.7.2 Dimensions and Deformations

SD590, SD685, and SD785 bars are manufactured in sizes 5.5 mm to 50 mm diameters with deformations similar to those specified in the Japanese Standard JIS G 3112, *Steel Bars for Concrete Reinforcement*. These bars exhibit deformations comparable to those of ASTM A615 and A706 bars.

4.8 AS/NZS 500E REINFORCEMENT

Members resisting earthquake effects in Australia and New Zealand are reinforced with AS/NZS high strength Grade 500E. The E at the end of the grade designation indicates that it is meant to be used in earthquake-resistant members. Researchers have used Grade 500E reinforcement in cyclically tested members and joints, and practitioners have specified it for existing structures. Australian/New Zealand Standard 4671:2001 specifies the requirements for this reinforcement. Microalloying or quenching and tempering are used to make it. Figure 4.6 shows representative stress-strain curves.

Tensile strength properties of Grade 500E reinforcement are provided in Table 4.9. There are both upper and lower limits on the yield strength, R_e, and the ratio of tensile strength to yield strength (R_m/R_e). Elongation is reported as uniform elongation, A_{gt}, measured in accordance with ISO 15630-1, which is the elongation taken from extensometer readings corresponding to the maximum force applied to the bar during tensioning. Alternatively, plastic elongation is measured after failure between gage marks, which are set before testing and are spaced at intervals up to 25 mm. If necking occurs between two gage marks, that span is not used for measuring plastic elongation.

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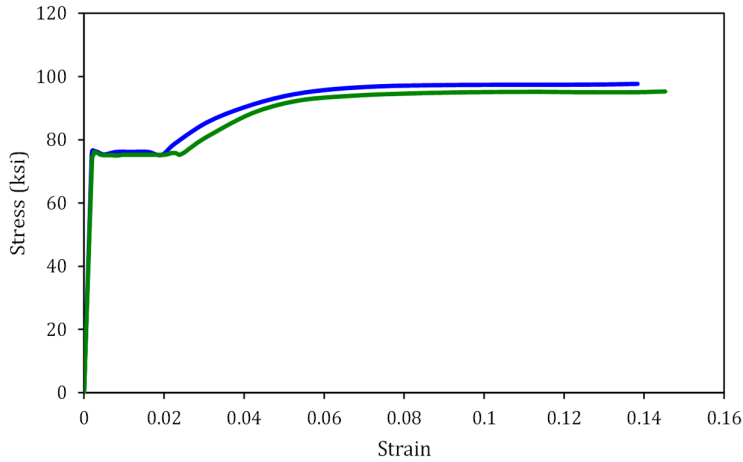

FIGURE 4.6 Representative stress-strain curves for AS/NZS Grade 500E (72.5 ksi) reinforcement (Lin *et al.* 2000).

TABLE 4.9
Tensile properties of AS/NZS 500E reinforcement (AS/NZS 4671:2001)

Property	Requirement
Yield strength[1,2], R_e	$R_{ek,L}$ = 72.5 ksi (500 MPa) ≤ R_e ≤ $R_{ek,U}$ = 87 ksi (600 MPa)
Tensile strength-to-yield strength ratio, R_m/R_e	$1.15 \leq R_m/R_e \leq 1.40$
Uniform elongation, A_{gt}	≥10%
Bending and rebending, $d_b \leq 16$ mm (0.63 in.)	Mandrel diameter = $4d_b$; bend angle = 90°
Bending and rebending, $d_b \geq 20$ mm (0.79 in.)	Mandrel diameter = $4d_b$; bend angle = 180°

[1]The AS/NZ Standard uses the 5% fractile concept, in which a small percentage of tests are allowed to be below the minimum strength.
[2]If an obvious yield point is not present, R_e corresponds to yield strength using the 0.2% offset method or the stress for a total elongation of 0.5%, whichever is smaller.

AS/NZS 4671:2019 also specifies higher strength grades 600 N and 750 N with minimum yield strengths of 600 MPa and 750 MPa. These bars have an ultimate-to-yield strength ratio in the range of 1.08 to 1.04. Minimum uniform elongation is specified within 4–5%. These bars are not meant to be used in seismic applications.

4.9 SAS 670 REINFORCEMENT

SAS 670 is a German-made high-strength threadbar reinforcing steel that comes in ductility Class B in accordance with EN 1992-1-1 (EC 2). The steel is quenched and

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tempered to increase its strength. It has been tested for use as longitudinal reinforcement for columns and has been used in geotechnical applications such as piles, soil nails and anchors (Bachmann *et al.* 2008). SAS 670 for use as column and wall longitudinal reinforcement in buildings with low seismic risk, such as seismic design categories A and B, an International Code Council Evaluation Service (ICC-ES) report (ICC 2009) has been issued. SAS 670 beams have also been cyclically tested (Lepage *et al.* 2012).

4.9.1 Tensile Properties

The required yield strength of SAS 670 reinforcement is 97 ksi. Tensile properties are included in Table 4.10, and a representative stress-strain curve is shown in Figure 4.7. The minimum ratio of tensile strength to yield strength of 1.1 is significantly less than what is required for ASTM A706 reinforcement (a ratio of 1.25).

TABLE 4.10
Tensile properties of SAS 670 reinforcement (Falkner *et al.* 2008)

Property	Requirement
Characteristic yield strength[1, 2], R_e	97 ksi (670 MPa)
Characteristic tensile strength[1], R_m	116 ksi (800 MPa)
Tensile-to-yield ratio, R_e/R_m	≥1.10
Total elongation under maximum load (uniform elongation), A_{gt}	≥5.0%
Bendability	#6 to #14 (bar diameters ≤ 43 mm)

[1]Using the 5% fractile concept, in which a small percentage of tests are allowed to be below the minimum strength.
[2]R_e corresponds to yield strength using the 0.2% offset method.

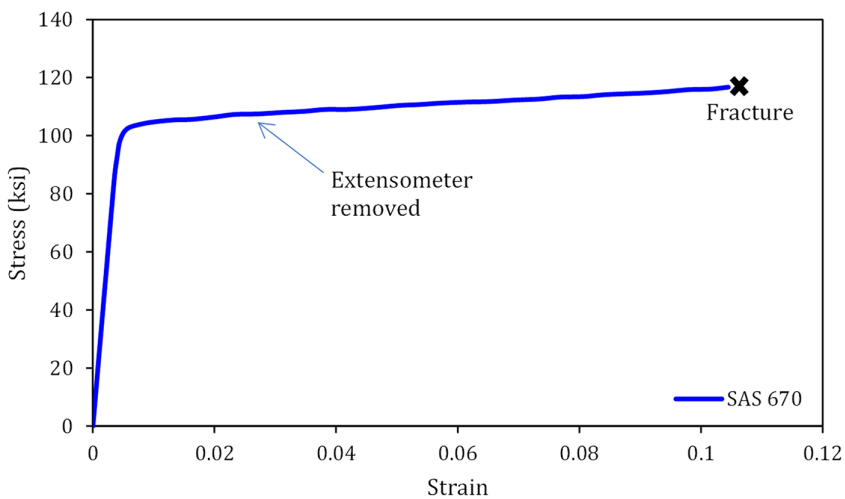

FIGURE 4.7 Representative stress-strain curve for SAS 670 reinforcement (NIST 2014).

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4.9.2 Dimensions and Deformations

SAS 670 bars are available in diameters ranging from 18 mm (0.71 inches) to 75 mm (2.95 inches). Surface deformations are in the form of right-hand threads that facilitate the easiest use of couplers.

4.10 HIGH STRENGTH REBAR IN ACI 318

Different grades of steel are used in reinforced concrete. Limitations on the minimum yield strength, ultimate strength and elongation are mentioned in ASTM specifications for reinforcing steel bars. ACI 318-19 (also Bangladesh National Building Code – BNBC 2020) allows the use of rebar meeting the following ASTM specifications:

a. ASTM A615 – Carbon steel
b. ASTM A706 – Low-alloy steel
c. ASTM A996 – Axle steel and rail steel

In addition, ACI 318-19 allows the use of high-strength low-carbon chromium steel specified by ASTM A1035 and stainless steel specified by ASTM A995.

4.10.1 Tensile Strength Requirements

In reinforced concrete applications, we are mainly concerned with the standards A615 and A706. The other standards are rarely applicable. Though ACI allows both these specifications, A706 is now the preferred standard in the United States, especially for seismic applications. The mechanical strength requirements of A615, A706 and A1035 specifications are shown in Tables A.5, A.6 and A.7 in the Appendix. It can be seen that the A615 specification now supports five different grades, namely Grades 40, 60, 75, 80 and 100. Grade 80 and Grade 100 were added to the specification in 2009 and 2015, respectively. The presence of Grade 75 now seems redundant. However, it is still retained for the convenience of the designers and construction industries for a smooth transition in the practice of using Grade 80 instead of Grade 75. Unlike A615, the specifications of A706 now allow only two grades – namely Grade 60 and Grade 80. In broad terms, ASTM A706 is more demanding and includes requirements for controlled tensile properties and restrictions on chemical composition, while ASTM A615 does not include comparable requirements. Tensile requirements of these specifications are given in Tables A.5 and A.6 in the Appendix.

For Grade 80 rebar, ASTM A706, like other specifications for non-prestressed steel reinforcement, prescribes a minimum yield strength of 80,000 psi (550 MPa) and a minimum tensile strength of 100,000 psi (690 MPa). The specification, however, also includes two limits or controls on tensile properties: 1) the actual yield strength cannot exceed 98,000 psi (675 MPa); and 2) actual tensile strength cannot be less than 1.25 times the actual yield strength. The latter requirement controls when the actual yield strength obtained in a laboratory tensile test of a bar is greater than 80,000 psi (550 MPa). If laboratory tension tests of reinforcing bars resulted

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in measured yield strengths ranging from slightly above 80,000 psi (550 MPa) to the maximum permitted value of 98,000 psi (675 MPa), the corresponding tensile strengths would have to range from slightly above 100,000 to 122,500 psi (690 to 845 MPa) to meet the requirements for A706 Grade 80 bars. A similar criterion also holds true for Grade 60 bars.

4.10.2 Elongation and Bend Requirements

Another important characteristic of MS rebar is its ductility which is generally indicated by two parameters: (1) elongation at failure and (2) bendability. These two parameters govern the usability of rebar in seismic applications. The energy dissipation capability of rebar is generally indicated by its elongation value – the higher, the better. From Table A.5 and A.6 in the Appendix, it can be observed that the elongation requirement of the A706 rebar is much more stringent than that of the A615 rebar. For example, for up to No. 6 Grade 60 rebar, the A615 requires only 9% elongation at failure, while A706 specification requires at least 14% elongation. Similarly, for Grade 80 rebar, the minimum elongation requirement for up to No. 8 bars is 7% and 12% in A615 and A706 specifications, respectively.

The bendability requirements are summarized in Tables A.9, A.10 and A.11 in the Appendix where it can be observed that the pin diameters for bending are generally smaller in A706 specification. Also, for No. 14 and No. 18 bars, the bend angle is 90° in A615 while it is 180° for all sizes in A706. Thus, the bend test requirements of A706 specification are more stringent than that of A615 specifications. Bar bending requirements of ACI 318-19 as well as BNBC 2020 are shown in Table 4.11. The minimum diameter of the bend applies to both 90° and 180° hooks. Comparing the minimum diameter of Table 4.11 with that of Tables A.9, A.10 and A.11, it can be said that ASTM bend diameters are smaller than that of ACI/BNBC. It is to be noted that ACI bend specifications for hooks come from detailing requirements, while those of ASTM A706 or A615 are meant for quality control purposes.

TABLE 4.11
Minimum bend requirement in ACI/BNBC

Bar size	Minimum diameter of bend
10 mm ≤ d_b ≤ 25 mm	$6d_b$
25 mm < d_b ≤ 40 mm	$8d_b$
40 mm < d_b ≤ 57 mm	$10d_b$

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QUESTIONS

4-1. Give a brief historical account of the development of high-strength rebar around the world.

4-2. What are the characteristic differences in tensile and elongation properties of rebar in ASTM A615 and A706 standards?

4-3. How do ASTM A1035 bars differ from A615 or A706 bars in terms of tensile, bend and weldability characteristics?

4-4. Describe tensile, elongation and bend characteristics of ISO 6935 rebar. How do different ductility classes of ISO standard compare with the ASTM A706 standard?

4-5. Provide a brief overview of tensile strength properties for Japanese reinforcing steels. What unique specification of Japanese reinforcing steel make it different from other standards?

4-6. Discuss how ACI 318 building code adopts high-strength rebar standards such as ASTM A615, A706, etc. for concrete structures.

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5 Code Application of High Strength Reinforcing Bars

5.1 INTRODUCTION

According to ATC 115, a high-strength reinforcing bar (HSRB) is any reinforcing bar with a yield strength greater than a nominal 415 MPa (ATC 2014). Reinforcement bars with a yield strength of 415 MPa (Grade 60) have been used for most beams, girders, and columns in reinforced concrete construction over the last few decades, with reinforcement bars with a yield strength of 500 MPa being used less frequently for columns that are not part of a special moment resisting frame. However, in the construction of high-rise concrete buildings and structures, high-strength reinforcement bars are now replacing Grade 60 bars, particularly in seismically vulnerable areas. In earthquake-resistant buildings in Japan, reinforcement bars with yield strengths of up to 690 MPa are currently being used. Several investigations have been carried out to ascertain whether stronger reinforcing bars should be used in beams, girders, and columns that support both live and dead loads. The demand for higher strength reinforcing bars in seismic and non-seismic applications is growing, according to these studies. There are several advantages to using high-strength reinforcement bars in concrete construction. A reduction in reinforcement congestion, cost savings, and construction time savings are just a few of the advantages.

ACI 318 undergoes periodic updates to include requirements for reinforcing bars with higher strengths as new products enter the market. The 2019 version of *ACI Building Code Requirements for Structural Concrete and Commentary* (ACI 318-19) included a significant expansion of the permissible applications of high-strength steel reinforcement. High-strength steel reinforcement is defined as reinforcement that has a yield strength of 80 ksi (550 MPa) or 100 ksi (690 MPa). The most recent requirements and limitations for non-prestressed deformed reinforcement and non-prestressed plain spiral reinforcement, respectively, can be found in Tables 20.2.2.4a and 20.2.2.4b of ACI 318-19. Table 5.1 lists the currently available reinforcing bar grades, along with their respective minimum yield strengths and minimum tensile strengths. The information that is presented in the table has been obtained from the relevant ASTM specifications.

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TABLE 5.1
Currently available reinforcing bar grades

ASTM designation and type		Available grades	Minimum yield strength, f_y (psi)	Minimum tensile strength, f_u (psi)
A615, Carbon-steel		40	40,000	60,000
		60	60,000	90,000
		75	75,000	100,000
		80	80,000	105,000
		100	100,000	115,000
A706, Low-alloy steel[a]		60	60,000	80,000
		80	80,000	100,000
A955, Stainless-steel		60	60,000	90,000
		75	75,000	100,000
A996	Rail-steel	50	50,000	80,000
		60	60,000	90,000
	Axle-steel	40	40,000	70,000
		60	60,000	90,000
A1035[b], Low-carbon, chromium	CL	100	100,000	130,000
		120	120,000	150,000
	CM, CS	100	100,000	150,000
		120	120,000	150,000

[a]Maximum yield strengths are 78,000 psi and 98,000 psi for grades 60 and 80, respectively.
[b]Chemical compositions for the different alloy types are given in ASTM A1035.

5.2 BENEFITS

When compared to members reinforced with Grade 60 or lower bars, the use of HSRB in concrete members may result in smaller bar sizes and/or a lower total number of bars. This is in contrast to members reinforced with lower grades of bars. It is also possible that it will permit smaller member sizes. The following are a few examples that can be accomplished by specifying HSRB:

a. Increased efficiency in the placement and consolidation of the concrete.
b. Smaller member dimensions.
c. More space that can be utilized for other purposes.
d. Less cost for placement.
e. Reduced congestion, particularly around the joints.

Consider the column of reinforced concrete depicted in Figure 5.1a. The cross-sectional size is limited to 18 inches for architectural reasons. Assuming the column is

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not slender and is subjected to a concentric factored axial load P_u = 1,700 kips, the required longitudinal reinforcement for Grade 60 is 12 #9 bars (ρ = 3.7%) with f_c' = 10,000 psi. This is a relatively high reinforcement ratio and at lap splice locations, ρ = 7.4%, which is close to the code-allowed maximum of 8%. In addition, the clear distance between the longitudinal bars is approximately 3 inches. The combination of a high reinforcement ratio and a relatively narrow clear spacing may result in significant congestion at the joint itself. If Grade 80 bars were used instead (see Figure 5.1b), eight #9 bars would be needed (ρ = 2.5%). Not only is the reinforcement ratio more reasonable, but the likelihood of congestion at the joints is greatly reduced as well (the clear space between the bars is almost 5.5 inches).

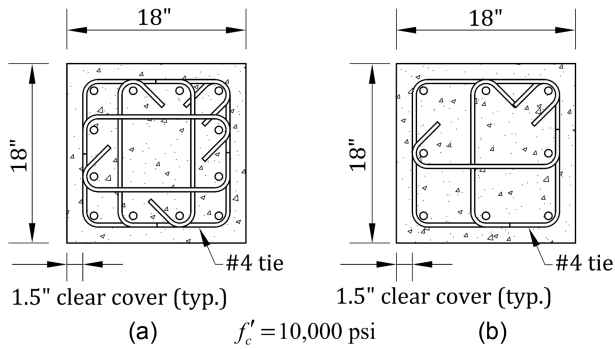

FIGURE 5.1 Example of reinforced concrete column. (a) Grade 60 reinforcement (b) Grade 80 reinforcement (CRSI 2016).

Figure 5.2 depicts a comparison of the interaction diagrams of the column with various bar grades. A column with high-strength longitudinal bars may have a smaller cross-sectional area than one with Grade 60 longitudinal bars, resulting in more usable space. Figure 5.3 depicts how the total area of a column, A_g, decreases as a function of the yield strength, f_y, for various concrete compressive strengths, f_c', and the longitudinal reinforcement amounts A_{st}. It is evident from the figure that the combination of high-strength reinforcement and high-strength concrete has the greatest impact on reducing the required column area for a given percentage of longitudinal reinforcement A_{st}/A_g. When reducing the cross-section of reinforced concrete members, it is always important to consider the possibility of congestion issues that may result.

5.3 DESIGN ISSUES

Design professionals can design reinforced concrete structures with HSRB using the requirements and limits prescribed in ACI 318-19, which are summarized in Table 5.2 for deformed bars. It is important to note that some provisions of ACI 318 may need adjustment before HSRB can be used in applications where the yield

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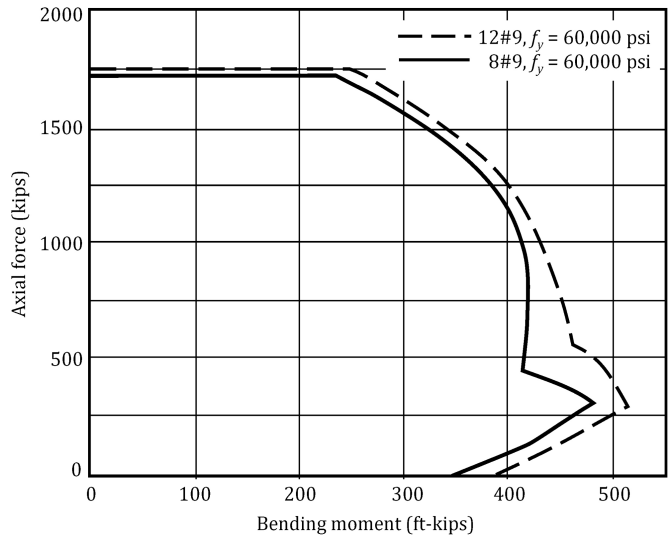

FIGURE 5.2 Interaction diagrams of the columns in Figure 5.1 with 12 #9 bars (Grade 60) and 8 #9 bars (Grade 80) (CRSI 2016).

strength is greater than the limits prescribed in Table 5.2 (for example, using longitudinal tension reinforcement with a yield strength of 100,000 psi in beams). As new research and/or new analytical studies become available related to the use of HSRB, it is anticipated that ACI 318 will be updated accordingly. For example, as discussed previously, ACI Committee 318 has considered the general use of Grade 80 reinforcement in special seismic systems as well as some other possible modifications related to HSRB in the current edition of ACI 318.

A brief discussion is presented below on some design issues related to the use of HSRB and the relevant interpretation of ACI 318 provisions. The following list of issues and observations is not meant to be comprehensive, nor is it meant to discourage the use of HSRB. Rather, it is provided so that design professionals have a clearer understanding of the recent ACI 318 provisions and what additional information is required for the general use of HSRB beyond the limitations in Table 5.2. Some in-depth discussion on these and other issues can be found in ATC 115 (ATC 2014).

5.3.1 Strength

5.3.1.1 Flexure and Axial Load Strength

Compression-controlled sections are defined as those where the net tensile strain ε_t does not exceed the compression-controlled strain limit ε_{ty}, which is defined in ACI 21.2.2.1 as f_y/E_s. The current definition of ε_{ty} is based on a stress-strain curve that is linear-elastic to the yield plateau. For HSRB with a rounded stress-strain curve (see Figure 5.4) where f_y must be determined by the 0.2% offset method, the definition of ε_{ty} may need to be revised from f_y/E_s. In lieu of revising the definition, specific values

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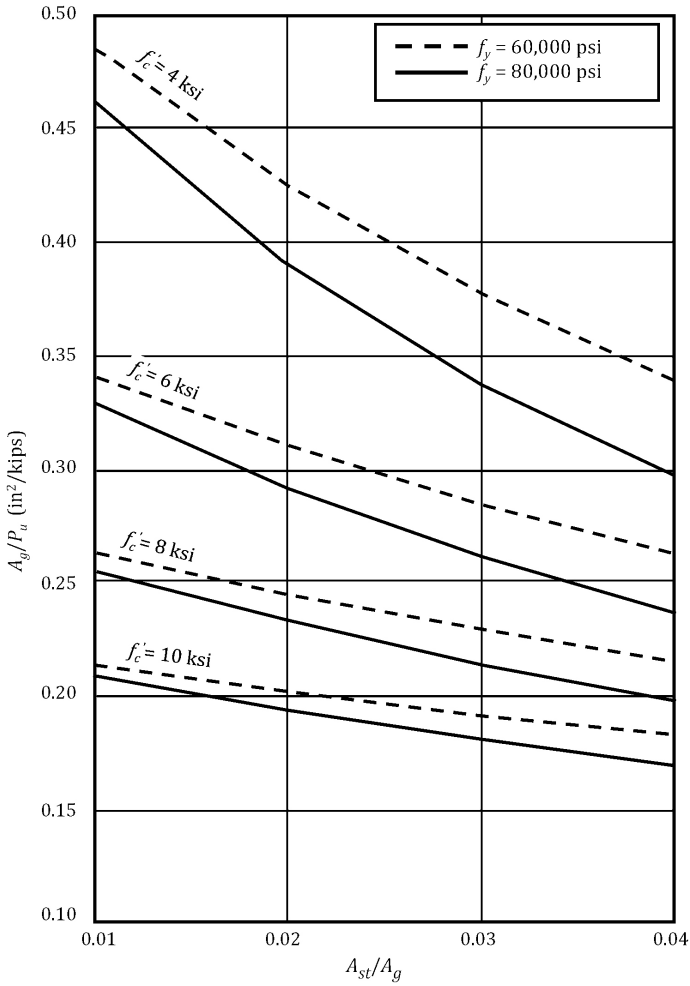

FIGURE 5.3 Required column area A_g as a function of reinforcing bar yield strength f_y, concrete compressive strength f_c', and amount of longitudinal reinforcement A_{st} (CRSI 2016).

of ε_{ty} can be provided for different grades, similar to prestressed reinforcement (see ACI 21.2.2.2).

Tension-controlled sections are defined as those where the net tensile strain ε_t is equal to or greater than the tension-controlled strain limit. Prior to ACI 318-19, the limiting value was 0.005, which is approximately 2.5 times the yield strain of 0.002 for ASTM A615 Grade 60 reinforcement. The yield strain will typically be larger for HSRB. To address this issue in ACI 318-19, the limit has been changed to $\varepsilon_{ty} + 0.003$ to provide levels of ductility consistent with the current provisions. The serviceability issues must be taken into account due to changes in this provision.

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TABLE 5.2
ACI 318 yield strength limitations for deformed reinforcing bars

Function	Application		Upper limit of f_y or f_{yt} permitted for design calculations, psi	Deformed bars
Flexure; axial force; and shrinkage and temperature	Special seismic systems	Special moment frames	80,000	A706[2]
		Special structural walls[1]	100,000	
	Other		100,000[3, 4]	A615, A706, A955, A996, A1035
Lateral tie of longitudinal bars; or confinement to concrete	Special seismic systems		100,000	A615, A706, A955, A996, A1035
	Spirals		100,000	A615, A706, A955, A996, A1035
	Other		80,000	A615, A706, A955, A996
Shear	Special seismic systems[5]	Special moment frames[6]	80,000	A615, A706, A955, A996
		Special structural walls[7]	100,000	
	Spirals		60,000	A615, A706, A955, A996
	Shear friction		60,000	A615, A706, A955, A996
	Stirrups, ties, hoops		60,000	A615, A706, A955, A996, A1035
			80,000	Not permitted
Torsion	Longitudinal and transverse		60,000	A615, A706, A955, A996
Anchor reinforcement	Special seismic systems		80,000	A706[2]
	Other		80,000	A615, A706, A955, A996
Regions designed using strut-and-tie method	Longitudinal ties		80,000	A615, A706, A955, A996
	Other		60,000	

[1] All components of special structural walls, including coupling beams and wall piers.

[2] ASTM A615 Grade 60 shall be permitted if the requirements of ACI 20.2.2.5(b) are satisfied.

[3] In slabs and beams that are not part of a special seismic system, bars that extend through or pass through special structural walls must meet ACI 20.2.2.5.

[4] In seismic design of intermediate and ordinary moment frames, $f_y > 80{,}000$ psi is not permitted for longitudinal reinforcement.

[5] In design calculations for diaphragms and foundations for seismic load effects as a part of special seismic system, shear rebar can have f_y or f_{yt} equal to 80 ksi maximum.

[6] In this application, shear reinforcement consists of stirrups, ties, hoops, and spirals in special moment frames.

[7] All lateral ties in special structural walls, coupling beams, and wall piers is included in this application. In coupling beams, diagonal bars must meet ASTM A706 or Footnote 2.

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For non-prestressed one-way slabs, two-way slabs, and beams, the net tensile strain ε_t was prescribed to be equal to or greater than 0.004 in the previous versions of the code to mitigate brittle flexural behavior in case of an overload (see ACI 7.3.3.1, 8.3.3.1, and 9.3.3.1, respectively). For HSRB, the elongation capacity ε_u is generally less than that for Grade 60 reinforcing bars. Therefore, the prescribed net-tensile strain has been generalized and given in Table 21.2.2 of ACI 318.

Current methods for calculating the flexural strength of one-way slabs, two-way slabs, and beams assume that the stress-strain curve of the reinforcement includes a yield plateau. For members reinforced with HSRB without a yield plateau, the current methods to determine flexural strength must be validated analytically and/or experimentally.

5.3.1.2 Shear Strength

Research has shown that the one-way and two-way shear strength of the concrete is influenced by the longitudinal reinforcement ratio: the larger the ratio, the larger the concrete shear strength. Using HSRB may result in a smaller required longitudinal reinforcement ratio, which may lead to reduced concrete shear strength.

For HSRBs that are utilized as shear reinforcement, crack control needs to be addressed in cases where the yield strength exceeds 60,000 psi.

5.3.2 Serviceability

5.3.2.1 Deflections and Cracking

Since higher strength bars undergo higher strain, careful consideration should be given while selecting the minimum member thicknesses following the provisions of ACI 7.3.1, 8.3.1, and 9.3.1 for one-way slabs, two-way slabs, and beams, respectively.

The expression for the effective moment of inertia, I_e of non-prestressed members in ACI 24.2.3.5, which is used in calculating deflections of reinforced concrete members, needs to be verified for members utilizing HSRB.

The time-dependent deflection factor λ_Δ in ACI 24.2.4.1.1 that is used in calculating long-term deflections is independent of the yield strength of the reinforcement in the member. This factor needs to be verified for HSRB.

5.3.2.2 Drift

Depending on a number of factors, there is a potential for increased flexural cracking to occur in reinforced concrete flexural members with HSRB. As such, a reduction in the flexural stiffness is anticipated, resulting in possible larger drift.

Increased cracking is not anticipated in columns with HSRB. However, the amount of longitudinal reinforcement and/or the size of the column may be decreased, which would have an impact on stiffness and drift.

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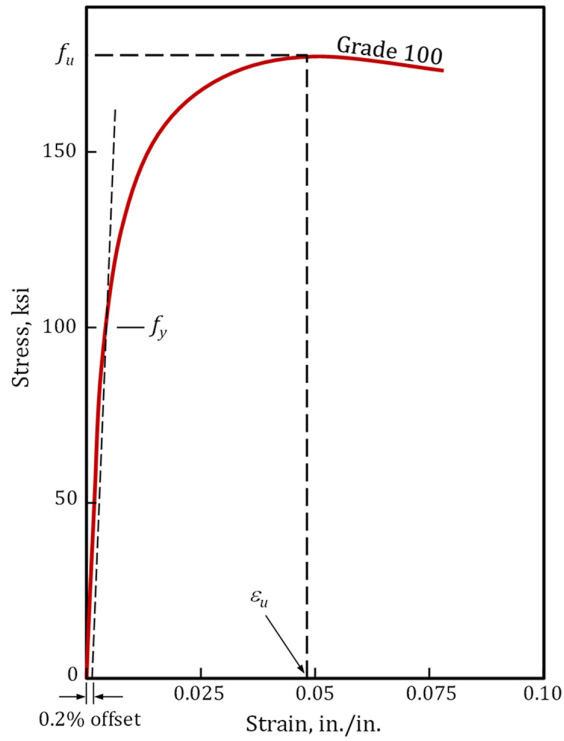

FIGURE 5.4 Rounded stress-strain curve for Grade 100 reinforcing bars (CRSI 2016).

5.4 BENEFITS OF GRADE 80 AND GRADE 100 REINFORCEMENT

Utilizing Grade 80 reinforcement in concrete members may result in the following,

a. Congestion is reduced, particularly at joints, because smaller bar sizes and/or fewer Grade 80 bars may be required when compared to members reinforced with Grade 60 bars.
b. Because of less congestion, concrete placement and consolidation can improve.
c. Lower placement costs as a result of fewer bars to be placed in the field.
d. Smaller member dimensions.
e. Smaller member sizes result in more usable space.

The information in Figure 5.3 shows how smaller member sizes can be achieved with Grade 80 reinforcement. The curves in this figure represent the required gross areas, A_g, for uniaxially loaded reinforced concrete columns based on concrete compressive strength, longitudinal reinforcement area, A_{st}, and reinforcement grade. As expected, the required A_g for a column with Grade 80 longitudinal bars is less than the same required for a column with Grade 60 longitudinal bars for a given steel

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ratio A_{st}/A_g. Combining Grade 80 reinforcement with high-strength concrete has the greatest impact on reducing the required column area, which increases usable space (CRSI 2016).

Given the aforementioned benefits and other considerations, it is expected that Grade 80 reinforcement will be utilized in buildings as follows:

a. In combination with high-strength concrete, as longitudinal reinforcement in columns designed in accordance with ACI Chapter 10, particularly in the lower stories of mid- and high-rise buildings.
b. In SDC D, E, and F, as longitudinal and transverse reinforcement in special moment frames.
c. In special structural walls, as longitudinal and transverse reinforcement.
d. Mat foundations supporting buildings assigned to SDC D, E, or F as flexural reinforcement.
e. All sorts of reinforcement in coupling beams that are part of special structural walls.
f. As the main rebar in collector elements in buildings assigned to SDC D, E, or F.

It is anticipated that Grade 100 reinforcement will be utilized in the following ways in buildings:

a. As longitudinal reinforcement in columns designed per ACI Chapter 10, particularly in the lower stories of high-rise buildings in conjunction with high-strength concrete.
b. As longitudinal wall reinforcement in SDC A, B, or C-designated buildings.
c. As flexural reinforcement in mat foundations supporting buildings designed as SDC D, E, or F.
d. As both the lateral and longitudinal reinforcement in special structural walls.
e. As diagonal, longitudinal, and transverse reinforcement in coupling beams comprising special structural walls.
f. As longitudinal reinforcement in collector elements assigned to SDC D, E, or F.

5.5 DUCTILITY

The magnitude of ultimate deformation (ε_u) on the stress-strain curve determines the ductility of reinforcing bars. The bend test is one method (see Sec. 2.2.2) for evaluating ductility; however, it is not a quantitative method for predicting service performance in all bending operations. The requirements for bend tests are specified in the ASTM specifications for the reinforcing bar grades. In accordance with this specification, bars are bent around a pin (or mandrel). The mandrel has a specified diameter. The bar is bent to a specified degree of bending. The bend diameter varies based on the diameter of the bar. The bar passes the bend test if there are no visible

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cracks on the outer radius of the bent portion. These ASTM requirements are prescribed for quality control of the rebar production system. Typically, ACI 318 prescribes bar bending requirements for rebar detailing. ACI 318 minimum inside bend diameters (ACI 25.3) are generally grade-independent and larger than the ASTM specifications. Bar ductility and structural ductility are two distinct issues that must be addressed when designing and detailing a reinforced concrete structure, particularly for those structures assigned to higher Seismic Design Categories (SDC).

5.6 SURFACE DEFORMATION REQUIREMENT

For all grades of deformed reinforcing bars conforming to the specifications of ASTM A706, a new requirement [ACI 20.2.1.3 (b) (iii)] was added:

> For all grades of ASTM A706 reinforcement, the radius at the base of each deformation shall be at least 1.5 times the height of the deformation. This requirement applies to all deformations, including transverse lugs, longitudinal ribs, grade ribs, grade marks, and intersections between deformations…

The magnitude of the strain localization at the juncture between the deformation and the barrel of the bar is affected by the ratio *r*/*h* (see Figure 5.5). Inelastic fatigue and tensile fracture (ductility) performance suffer from increased strain localization. This restriction is in place to prevent inelastic fatigue cracks from forming at the re-entrant corner of the root of the deformation. This is a critical design issue for seismic loading.

When specifying Grade 80 and Grade 100 reinforcing bars for a project conforming to ASTM A615 and ASTM A706, it is critical to include these new property requirements, as well as the corresponding ASTM requirements, in the project specifications.

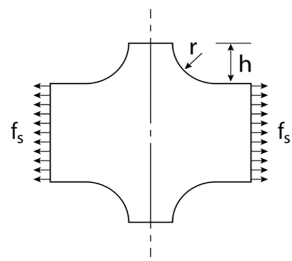

FIGURE 5.5 Radius and height of deformation on a deformed reinforcing bar (CRSI 2020a).

5.7 APPLICATIONS OF GRADE 80 REINFORCING BARS

ACI Table 20.2.2.4 (a) lists the types of deformed reinforcing bars that are permitted for specific structural applications. Grade 80 reinforcement can now withstand the following forces:

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a. In buildings assigned to Seismic Design Categories (SDC) D, E, and F, ASTM A706 Grade 80 rebar can be used to resist flexure, axial forces, and shear forces due to gravity and earthquake loads required in special moment frames (previously, 60 ksi was the maximum permitted value of the yield strength in such applications).
b. Gravity and earthquake loads causing flexure, axial forces, and shear forces in special structural walls, which are required to resist in buildings assigned to Seismic Design Categories (SDC) D, E, and F (previously, 60 ksi was the maximum permitted yield strength in such applications).

Table 5.3 lists the permitted uses and applications of Grade 80 reinforcement. Because the maximum values of f_y and f_{yt} are given in Table 5.3 in conformity with

TABLE 5.3
Usages and applications of Grade 80 deformed reinforcement in ACI 318-19

Usage	Application	Maximum value of f_y and f_{yt} permitted for design calculations psi[1]	Deformed bars
Flexure Axial force	Special moment frames	80,000 (60,000)	A706[2]
Shrinkage and temperature	Special structural walls[3]	100,000 (60,000)	A706[2]
	Other	100,000 (80,000)	A615, A706, A955, A996, A1035
Lateral support of longitudinal bars	Special seismic systems[4]	100,000	A615, A706, A955, A996, A1035
Concrete confinement	Spirals	100,000	A615, A706, A955, A996, A1035
	Others	80,000	A615, A706, A955, A996
Shear	Special moment frames[5]	80,000 (60,000)	A615, A706, A955, A996
	Special structural walls[6]	100,000 (60,000)	A615, A706, A955, A996
Anchor reinforcement	Special seismic systems[4]	80,000	A706[2]
	Other	80,000	A615, A706, A955, A996
Regions designed using strut-and-tie method	Longitudinal ties	80,000	A615, A706, A955, A996

[1]The permitted values in ACI 318-14 are shown in parentheses.
[2]Provided that the requirements of ACI 20.2.2.5b are satisfied, it is permitted to use ASTM A615 Grade 60 bars instead.
[3]All components of special structural walls, including coupling beams and wall piers are included.
[4]Including special moment frames as well as special structural walls.
[5]Stirrups, ties, hoops, and spirals in special moment frames are included in this application.
[6]All transverse reinforcement in special structural walls, coupling beams, and wall piers are included. Diagonal bars in coupling beams shall meet ASTM A706 specification or Footnote 2 in this table.

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ACI Table 20.2.2.4 (a), ASTM A996 bars with a maximum yield strength of 60,000 psi and ASTM A1035 Grade 100 bars are included in the table, despite the fact that these two types of deformed bars do not have a yield strength of 80,000 psi. Applicable limits in the previous version of the code, ACI 318-14, are shown in numbers in parentheses below the tabulated values of f_y and f_{yt}. For concrete confinement and lateral support of longitudinal bars, the permitted values of f_y and f_{yt} are the same as those specified in ACI 318-14.

In special structural walls, ASTM A615 Grade 60 reinforcement may be used instead of ASTM A706 Grade 80 reinforcement if the requirements in ACI 20.2.2.5 (b) are met as specified in Footnote [2] in ACI Table 20.2.2.4 (a). In ACI 318-19, updated requirements are given in ACI 20.2.2.5 (b). In special seismic systems, such as special moment frames and special structural walls, ASTM A615 Grade 80 rebar is not allowed to resist earthquake-induced moments, axial forces, or both; or be used as anchor reinforcement [see ACI 20.2.2.5 (b)] in Seismic Design Categories C, D, E and F.

In special seismic systems and all other systems, Grade 80 plain spiral reinforcement (ASTM A615, ASTM A706, and ASTM A955) is permitted to be used as lateral support of longitudinal bars or as concrete confinement for members [ACI Table 20.2.2.4 (b)]. This is the same requirement as in ACI 318-14.

5.8 APPLICATIONS OF GRADE 100 REINFORCING BARS

ACI Table 20.2.2.4 lists the types of deformed reinforcing bars that are permitted for specific structural applications. Grade 100 reinforcement can now handle the following forces:

a. Gravity and wind loads causing flexure and axial forces (in such applications, the maximum permitted value of the yield strength was 80,000 psi in the earlier specification).
b. Gravity and earthquake loads causing flexure, axial forces, and shear forces in special structural walls in buildings assigned to Seismic Design Categories (SDC) D, E, and F (the maximum permitted yield strength was 60,000 psi in the earlier specification).

Table 5.4 lists the permitted uses and applications of Grade 100 reinforcement. Note that maximum f_y and f_{yt} values are given in Table 5.4 in accordance with ACI Table 20.2.2.4 (a); thus, ASTM A996 and ASTM A955 are included in the table despite the fact that these two types of deformed bars do not have a yield strength of 100 ksi. The numbers in parentheses below the tabulated values of f_y and f_{yt} were the maximum permitted yield strengths in previous versions of the code, such as ACI 318-14. For lateral support of main bars and concrete confinement, the permitted values of f_y and f_{yt} are unchanged from ACI 318-14.

According to ACI Table 20.2.2.4 (a), Footnote 2, ASTM A615 Grade 60 reinforcement may be used in special structural walls instead of ASTM A706 Grade 100 reinforcement if the requirements in ACI 20.2.2.5 (b) are met (the requirements in ACI 20.2.2.5 (b) have been amended in ACI 318-19). In special seismic systems (that is,

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in special moment frames and special structural walls; see ACI 20.2.2.5 (b)), ASTM A615 Grade 100 (and ASTM A615 Grade 80) reinforcement is not permissible.

In special seismic systems and all other systems, Grade 100 plain spiral reinforcement (ASTM A615, ASTM A706, and ASTM A1035) is permitted to be utilized as lateral support of longitudinal bars or as concrete confinement for members (ACI Table 20.2.2.4 (b)). These specifications are identical to those in ACI 318-14.

TABLE 5.4
Usages and applications of Grade 100 deformed reinforcement in ACI 318-19

Usage	Application	Maximum value of f_y and f_{yt} permitted for design calculations psi[1]	Deformed bars
Flexure Axial force	Special structural walls[2]	100,000 (60,000)	A706[3]
Shrinkage and temperature	Other[4]	100,000 (80,000)	A615, A706, A955, A996, A1035
Lateral support of longitudinal bars	Special seismic systems[5]	100,000	A615, A706, A955, A996, A1035
Concrete confinement	Spirals	100,000	A615, A706, A955, A996, A1035
Shear	Special structural walls[6]	100,000 (60,000)	A615, A706, A955, A996

[1]Number in parentheses indicate the permitted values of f_y and f_{yt} in ACI 318-14.
[2]For all components of special structural walls, including coupling beams and wall piers.
[3]If the requirements of ACI 20.2.2.5b are met, ASTM A615 Grade 60 is allowed to be used instead.
[4]Structural elements of special moment frames are excluded from this category. Also, Grade 100 reinforcement for main longitudinal rebar is not permitted to resist seismic forces in intermediate moment frames and ordinary moment frames.
[5]Applies to special moment frames and special structural walls.
[6]All transverse reinforcement in special structural walls, coupling beams, and wall piers are included. Diagonal bars in coupling beams must comply with ASTM A706 or Footnote 3 in this table.

5.9 DEVELOPMENT AND SPLICE LENGTHS

In order for a concrete flexural member to operate as anticipated in line with the strength design approach, flexural reinforcement must be correctly developed or anchored. The concepts of flexural bond and anchorage bond were utilized to establish the required lengths of flexural reinforcing bars prior to the 1971 code. To avoid bond failure or splitting, the calculated tension force in every bar at any section had to be developed on either side by adequate embedment length and anchorage or hooks.

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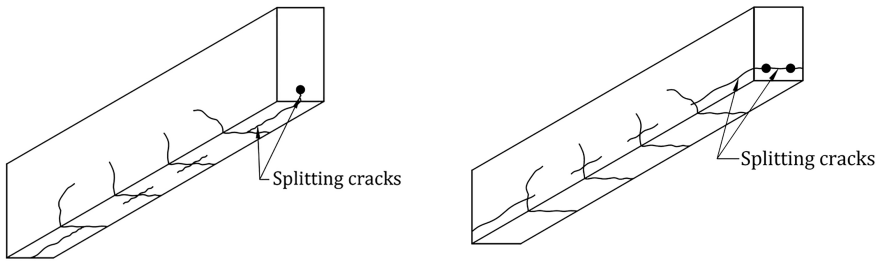
The idea of development length was initially introduced in the 1971 version of ACI 318 code and was based on average bond stress over the reinforcement's embedment length. Take a look at the reinforcing bar close to the bottom of the beam in Figure 5.6 (a). The concrete-steel bond stress changes along the length of the member, and its magnitude is governed by the stress in the bar and the occurrence of cracks. The relatively thin concrete that is confining the bar can break when subjected to severe loads caused by external bending moments, as depicted. The ribs of the distorted bars are bearing against the concrete, which causes the splitting. Complete bond failure happens when the splitting extends to the end of a bar that is not correctly anchored, and the beam can no longer resist the applied bending moment as specified. This type of failure can be prevented if the bar is long enough (or adequately anchored).



(a) Splitting of concrete along a single bar of flexural reinforcement.

(b) Splitting of concrete along multiple bars of flexural reinforcement.

FIGURE 5.6 Splitting of concrete in beams due to excessive tension in rebar (Fanella 2011).

Horizontal splitting along the plane of the reinforcing bars can also occur in the more usual circumstances where more than one reinforcing bar is present in the section (see Figure 5.6 (b)). Flexure and/or shear cause this sort of splitting to start at a diagonal crack. In this situation, the minimum development length of the bars required would be more than that of the single bar depicted in Figure 5.6 (a).

The concept of development length can be expressed simply as follows: in order to properly develop the bars, minimum lengths or extensions of reinforcement must be provided beyond the regions of peak stress in the reinforced element. These peak stress regions are sometimes referred to as critical sections, and they occur at points of maximum stress (maximum bending moment) and where adjacent reinforcement is ended. Both sides of a critical segment require development length or reinforcement anchorage.

In addition to embedment length, development or anchorage can be achieved with hooks, headed deformed bars, mechanical devices, or a combination thereof. The following discussion focuses on the development of flexural reinforcement where deformed bars and deformed wire are in tension. The code provisions for the development length of reinforcing bars in tension must be examined prior to the general requirements for flexural reinforcement.

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5.9.1 Development Length for Bars and Wires in Tension

According to ACI 318-19, the equation for development length for bars in tension shall be calculated based on the expressions given in Equation 5.1. However, in no case, the development length in tension shall be less than 12 inches.

$$l_d = \left(\frac{3}{40} \frac{f_y}{\lambda \sqrt{f'_c}} \frac{\Psi_t \Psi_e \Psi_s \Psi_g}{\left(\frac{c_b + K_{tr}}{d_b} \right)} \right) d_b \qquad 5.1$$

In the above equation, the term $(c_b+K_{tr})/d_b$ may not be taken greater than 2.5. Definition and values of different factors such as λ and Ψ can be found in Table 5.5.

TABLE 5.5
Modification factors for the development of deformed bars and deformed wires in tension

Modification factor	Condition	Value
Lightweight λ	Lightweight concrete	0.75
	Normal-weight concrete	1.0
Reinforcement grade Ψ_g	Grade 40 or Grade 60	1.0
	Grade 80	1.15
	Grade 100	1.3
Epoxy[1] Ψ_e	Epoxy-coated or zinc and epoxy dual-coated reinforcement with clear cover less than $3d_b$ or clear spacing less than $6d_b$	1.5
	Epoxy-coated or zinc and epoxy dual-coated reinforcement for all other conditions	1.2
	Uncoated or zinc-coated (galvanized) reinforcement	1.0
Size Ψ_s	#7 and larger bars	1.0
	#6 and smaller bars and deformed wires	0.8
Casting position[1] Ψ_t	More than 12 in of fresh concrete placed below horizontal reinforcement	1.3
	Other	1.0

[1]The product $\Psi_t \Psi_e$ need not exceed 1.7.

The parameters c_b and K_{tr} are defined as follows.

c_b = spacing or cover dimension, inches.

Use the smaller of the distances between the center of the bar and the nearest concrete surface or 1.5 times the center-to-center spacing of the bars being used.

K_{tr} = transverse reinforcement index: $40A_{tr}/(sn)$.

Where A_{tr} = total cross-sectional area of all transverse reinforcement that is within the spacing s (Figure 5.7) and that crosses the potential plane of splitting through the reinforcement being developed, in^2.

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s = maximum spacing on centers of transverse reinforcement within l_d, inches.

n = number of bars or wires being developed along the plane of splitting.

As a simplification, the designer is permitted to use $K_{tr} = 0$ even if transverse reinforcement is present.

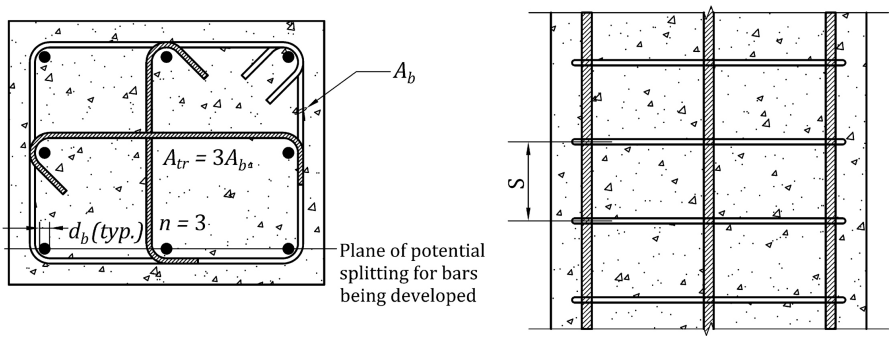

FIGURE 5.7 Transverse reinforcement index, K_{tr} (CRSI 2020a).

According to ACI Code 25.4.2.2, for reinforcement with $f_y \geq 80{,}000$ psi spaced closer than 6 inches to center, transverse reinforcement must be provided such that K_{tr} is not smaller than $0.5d_b$. According to ACI Code 9.7.1.4 and 10.7.1.3, however, K_{tr} must be at least $0.5d_b$ in all beams and columns where reinforcement with $f_y \geq 80{,}000$ psi is developed or spliced, independent of the bar spacing.

The limit of 2.5 on $(c_b+K_{tr})/d_b$ is imposed to avoid pullout failure. Note that in Equation 5.1 and in all other ACI Code equations relating to the development length and splices of reinforcement, values of $\sqrt{f'_c}$ are not to be taken greater than 100 psi because, as recognized in ACI Commentary 25.4.1.4, bond strength increases more slowly than $\sqrt{f'_c}$. The $\sqrt{f'_c}$, however, is reasonably accurate for values of f'_c up to 10,000 psi.

Instead of using Equation 5.1, simplified equations given in Table 5.6 can be used.

5.9.2 Development of Standard Hooks in Tension

Based on recent test results, the equation for determining the development length, l_{dh}, for deformed bars in tension terminating in a standard hook (Figure 5.8) has been changed, and now the expression (Equation 5.2) takes into account the effects of bar yield strength, spacing, and confinement by ties or stirrups (ACI 25.4.3.1). The outside bend radius of the hooks is also indicated in Figure 5.8.

$$l_{dh} = \left[\frac{f_y \Psi_e \Psi_r \Psi_o \Psi_c}{55\lambda\sqrt{f'_c}}\right] d_b^{1.5} \qquad 5.2$$

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TABLE 5.6
Simplified tension development length in bar diameters according to the ACI code

	#6 (#19) and smaller bars and deformed wires	#7 (#22) and larger bars
Clear spacing of bars or wires being developed or spliced ≥ d_b, clear cover ≥ d_b, and stirrups or ties throughout l_b not less than the Code minimum	$l_d = \left(\frac{f_y \Psi_t \Psi_e \Psi_g}{25\lambda\sqrt{f'_c}} \right) d_b$	$l_d = \left(\frac{f_y \Psi_t \Psi_e \Psi_g}{20\lambda\sqrt{f'_c}} \right) d_b$
Clear spacing of bars or wires being developed or spliced ≥ $2d_b$, and clear cover ≥ d_b	Same as above	Same as above
Other cases	$l_d = \left(\frac{3 f_y \Psi_t \Psi_e \Psi_g}{50\lambda\sqrt{f'_c}} \right) d_b$	$l_d = \left(\frac{3 f_y \Psi_t \Psi_e \Psi_g}{40\lambda\sqrt{f'_c}} \right) d_b$

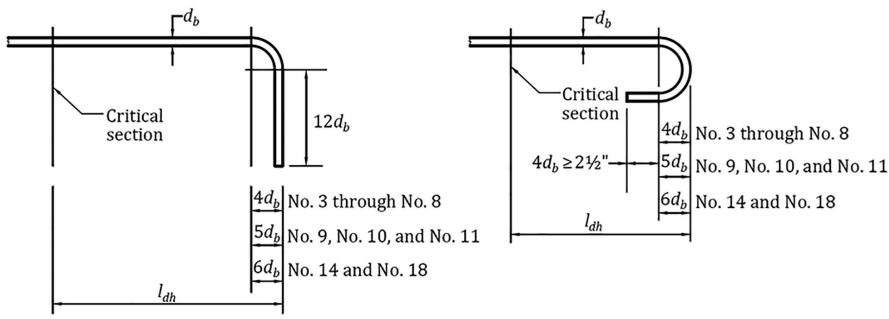

FIGURE 5.8 Development of standard hooks in tension (Darwin and Dolan 2021).

This equation is applicable to any f_y and the modification factors are given in Table A.13. Similar to the previous editions of ACI 318, l_{dh} must be the largest of that calculated by the above equation, $8d_b$, and 6 inches (ACI 25.4.3.1).

5.9.3 Development of Headed Deformed Bars in Tension

The equation for determining the development length, l_{dt}, for headed deformed bars (Figure 5.9) conforming to ACI 25.4.4.1 in tension, has been amended based on recent test results (ACI 25.4.4.2) as given in Equation 5.3.

$$l_{dt} = \left[\frac{f_y \Psi_e \Psi_p \Psi_o \Psi_c}{75\sqrt{f'_c}} \right] d_b^{1.5} \qquad 5.3$$

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This equation applies to rebar of any yield strength (earlier, yield strength had a maximum limit of 60,000 psi). The modification factors are given in Table A.14. Also, l_{dt} must be larger than that calculated from the above equation, $8d_b$, and 6 inches (ACI 25.4.4.2).

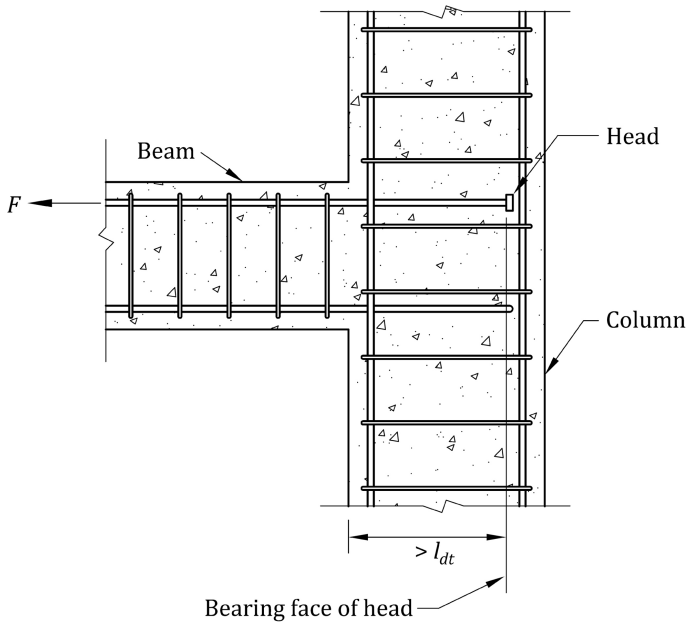

FIGURE 5.9 Headed deformed bar of beam extended to the far side of column core.

5.10 TWO-WAY SLABS

5.10.1 Grade 80 Rebar

ACI Table 8.3.1.1 now includes minimum thicknesses for non-prestressed two-way slabs without interior beams for Grade 80 reinforcement (see Table 5.7). The largest clear span in the floor system in inches is shown in Table 5.7. The minimum thicknesses for Grade 75 reinforcement in ACI 318-14 have been removed from this table.

5.10.2 Grade 100 Rebar

Where deflections are calculated in two-way slab systems without interior beams and with Grade 100 flexural reinforcement, the modulus of rupture, f_r, shall be taken equal to $5\sqrt{f'_c}$ instead of $7.5\sqrt{f'_c}$ when calculating deflection limits in ACI 8.3.2 (ACI 8.3.1.1; see also ACI 19.2.3.1). Slabs with Grade 100 main reinforcement may experience larger long-term deflections than those with Grade 60 longitudinal

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reinforcement unless service stresses calculated for cracked sections are smaller than 40,000 psi.

TABLE 5.7

Minimum thickness of non-prestressed two-way slabs without interior beams for Grade 80 reinforcement.[1]

Without drop panels[2]			With drop panels[2]		
Exterior panels		Interior panels	Exterior panels		Interior panels
Without edge beams	With edge beams[3]		Without edge beams	With edge beams[3]	
$l_n / 27$	$l_n / 30$	$l_n / 30$	$l_n / 30$	$l_n / 33$	$l_n / 33$

[1]The longest clear span in the floor system in inches is defined as l_n.

[2]ACI Section 8.2.4 defines drop panels.

[3]Exterior panels are considered to be without edge beams where $\alpha_f < 0.8$. The term α_f is the ratio of the flexural rigidity of a beam section to the flexural rigidity of a width of slab bounded laterally by center-lines of adjacent panels, if any, on each side the beam ($\alpha_f = E_{cb}I_b / E_{cs}I_s$).

5.11 BEAMS

The formulae in ACI 9.6.1.2 for determining the minimum area of flexural reinforcement, $A_{s,min}$, in non-prestressed beams, are the same as in previous editions of ACI 318, with the exception that in ACI 318-19, an upper limit of 80,000 psi on f_y has been imposed. $A_{s,min}$ must be calculated using f_y = 80,000 psi when Grade 100 longitudinal reinforcement is utilized in beams.

Specifications of ACI 9.7.1.4 for development lengths and lap splice lengths of longitudinal bars in beams have been revised when Grade 80 (and Grade 100) bars are used, which are essentially the same as those addressed previously in ACI 25.4.2.2 and ACI 25.5.1.5; the only difference is the center-to-center spacing limitation in those sections is not specified in ACI 9.7.1.4. Therefore, for Grade 80 and Grade 100 longitudinal bars, transverse reinforcement in beams must be provided such that $K_{tr} \geq 0.5d_b$, regardless of the bar spacing.

5.12 COLUMNS

When Grade 80 (and Grade 100) rebar is used in columns designed per ACI Chapter 10, the revised confinement requirements in ACI 10.7.1.3 for development lengths and lap splice lengths are identical to those for beams built per ACI Chapter 9. Transverse reinforcement is required along development and lap splice lengths such that $K_{tr} \geq 0.5d_b$.

For columns with Grade 100 longitudinal reinforcement subjected to axial compression, f_y must equal 80,000 psi when computing the maximum axial strength, $P_{n,max}$, as specified by ACI Table 22.4.2.1. This restriction on f_y is enforced because

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the compression capacity of unconfined concrete, which is estimated to occur at a strain limit of 0.003, is likely to be attained before the longitudinal bar stress reaches 80,000 psi. This restriction of 80,000 psi is not applicable when evaluating the strength of columns subjected to axial force and bending moment designed with Grade 100 longitudinal bars.

Figure 5.10 depicts the nominal and design strength interaction graphs for an 18-inch square tied column with four #9 longitudinal bars (Grade 100) and $f'_c = 4{,}000$ psi. Point A on the nominal strength interaction diagram indicates the nominal axial force, P_o, calculated by the ACI Equation 22.4.2.2 that corresponds to the limit of 80,000 psi for pure axial compression. This point is connected to the point where M_n=0 through a horizontal line. The interaction diagrams for the column with Grade 60 reinforcement are also shown for the purposes of comparison.

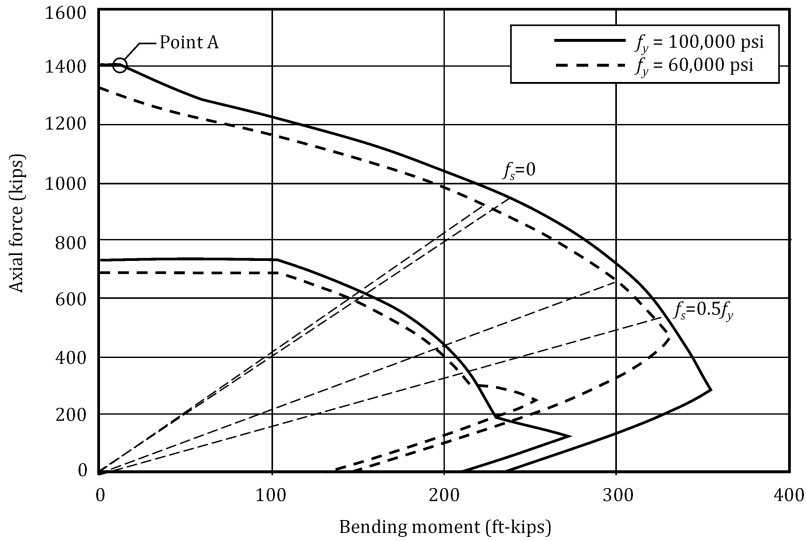

FIGURE 5.10 Design and nominal strength interaction diagrams for an 18-inch square column with Grade 100 longitudinal reinforcement (CRSI 2020a).

5.13 INTERMEDIATE MOMENT FRAMES

In buildings assigned to SDC C, new hoop spacing restrictions are specified in ACI 18.4.3.3 at the ends of columns that are part of intermediate moment frames. For columns with Grade 80 longitudinal bars, the maximum hoop spacing, s_o, over the length l_o is equal to the smaller of $6d_b$ of the smallest longitudinal bar enclosed, 6 inch, and one-half of the smallest cross-sectional dimension of the column (see Figure 5.11 and CRSI 2020c). The hoop spacing requirements in ACI 18.4.3.3 are also applicable to columns supporting reactions from discontinuous stiff members (ACI 18.4.3.6) and to the spacing of transverse reinforcement in beam-column joints (ACI 18.4.4.4).

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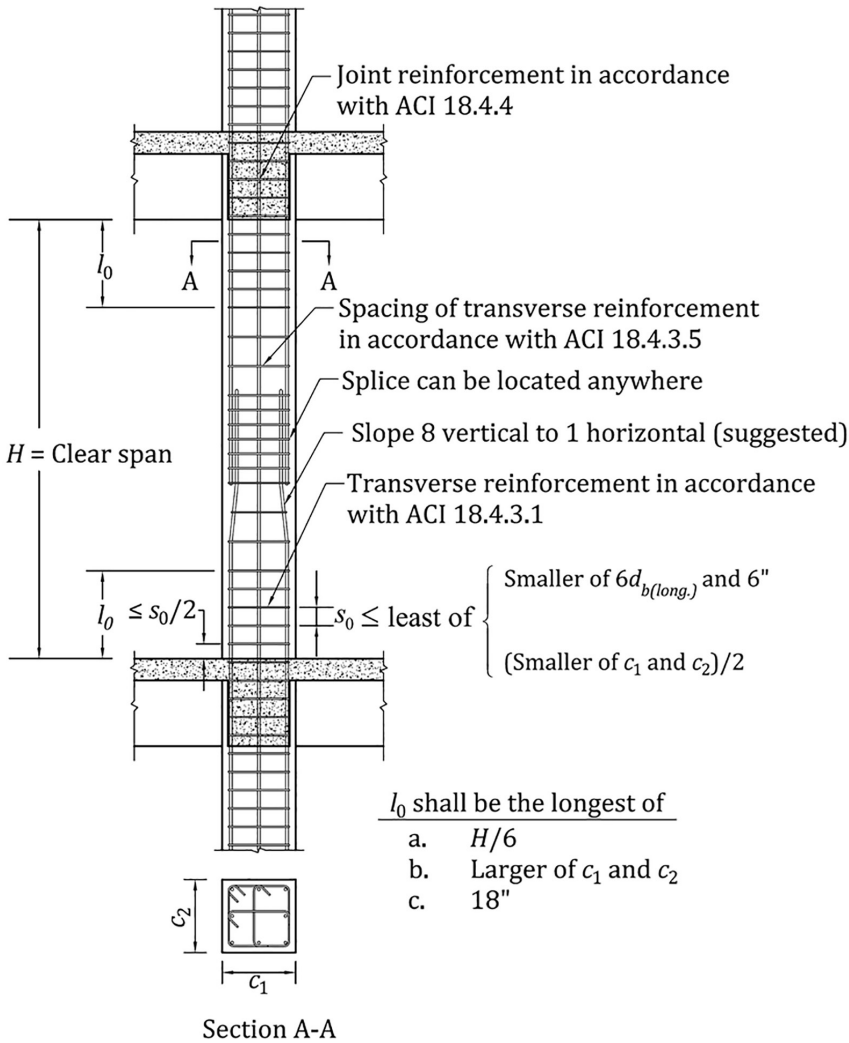
Joint reinforcement in accordance with ACI 18.4.4
l_0
A
A
Spacing of transverse reinforcement in accordance with ACI 18.4.3.5
Splice can be located anywhere
Slope 8 vertical to 1 horizontal (suggested)
H = Clear span
Transverse reinforcement in accordance with ACI 18.4.3.1
l_0 ≤ $s_0/2$
s_0 ≤ least of
Smaller of $6d_{b(long.)}$ and 6"
(Smaller of c_1 and c_2)/2
l_0 shall be the longest of
a. $H/6$
b. Larger of c_1 and c_2
c. 18"
c_2
c_1
Section A-A

FIGURE 5.11 Hoop spacing requirements for columns with Grade 80 longitudinal reinforcement in intermediate moment frames.

5.14 SPECIAL MOMENT FRAMES

5.14.1 Grade 80 Rebar

Grade 80 reinforcement is now permitted to resist flexure, axial forces, and shear in special moment frames, which are required in buildings allocated to SDC D, E, or F (ACI Table 20.2.2.4 (a)). Requirements for beams and columns with Grade 80 reinforcement in special moment frames are given in Tables 5.8 and 5.9, respectively. Flexural reinforcement requirements for beams in special moment frames

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with Grade 80 rebar are schematically shown in Figures 5.12 and 5.13. Transverse rebar requirements for columns in special moment frames with Grade 80 rebar are schematically shown in Figures 5.14 and 5.16.

5.14.2 Grade 100 Rebar

Due to the absence of adequate test data, Grade 100 reinforcement is not permitted to resist flexure and axial forces in special moment frames, which are required in buildings assigned to SDC D, E, or F (ACI Table 20.2.2.4 (a)).

It has been observed that the development lengths calculated in accordance with ACI 18.8.5.2 may come out larger than the development lengths for hooked bars calculated in accordance with ACI 18.8.5.1. Therefore, the tension development length requirements in ACI 18.8.5.2 for headed bars in joints of special moment frames, which are applicable to any f_y, may not be correct and shall need further clarification.

5.15 JOINTS OF SPECIAL MOMENT FRAMES

5.15.1 Grade 80 Rebar

Main beam reinforcement with Grade 80 rebar in a special moment frame shall extend through a beam-column joint for a length c_1, where c_1 is the depth of the joint parallel to the beam longitudinal reinforcement. The magnitude of c_1 must be greater than or equal to the following (ACI 18.8.2.3; see Figure 5.14) as given in Equation 5.4.

$$c_1 \geq \text{greateset of} \begin{cases} 12\,\text{inch} \\ 26d_b \text{ of the largest Grade 80 longitudinal bar} \\ h/2 \text{ of any beam framing into the joint generating} \\ \text{joint shear in the direction of analysis} \end{cases} \tag{5.4}$$

Joint transverse reinforcement must satisfy the requirements described in ACI 18.8.3. Normal weight concrete must be used in beam-column joints with Grade 80 longitudinal reinforcement in beams (ACI 18.8.2.3.1); ACI provisions do not permit lightweight concrete and Grade 80 longitudinal reinforcement because of the lack of sufficient test data.

5.15.2 Grade 100 Rebar

In special moment frame buildings assigned to SDC D, E, or F, Grade 100 reinforcement is not permitted to resist flexure and axial forces (ACI Table 20.2.2.4 (a)). The main reason for this is test data are not available for special moment frames with Grade 100 reinforcement.

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TABLE 5.8
Requirements for beams with Grade 80 reinforcement in special moment frames

Requirement	ACI section	Figure no.[1]
Maximum longitudinal reinforcement ratio: $\rho = 0.02$	18.6.3.1	4
Spacing, s, of hoops within a length equal to at least $2h$ from each end of the beam or on both sides of a section where flexural yielding is likely to occur: $s \le$ least of { $d/4$; 6 in.; $5d_b$ of the smallest primary flexural reinforcing bar[2] }	18.6.4.4	5
Beams with factored axial force $P_u > A_g f'_c / 10$.	18.6.4.7	-
Hoops satisfying ACI 18.7.5.2 through 18.7.5.4 must be provided along lengths given in ACI 18.6.4.1 Spacing, s, of hoops within a length equal to at least $2h$ from each end of the beam or on both sides of a section where flexural yielding is likely to occur $s \le$ least of { 6 in.; $5d_b$ of the smallest enclosed longitudinal beam bar }		

[1] See CRSI 2020c for in-depth design and detailing requirements for beams in special moment frames.
[2] Excludes longitudinal skin reinforcement required by ACI 9.7.2.3.

5.16 SPECIAL STRUCTURAL WALLS

ACI 18.10.6.5 specifies new maximum vertical spacing restrictions for transverse reinforcement at the edges of special structural walls, in building frame systems and dual systems in buildings classified as SDC D, E, or F (ACI 18.10.6.1). These limitations are prescribed to avoid bar buckling at the wall's ends. The maximum transverse reinforcement spacing in a wall with Grade 80 (or Grade 100) longitudinal reinforcement is smaller than in a wall with Grade 60 longitudinal reinforcement for a given longitudinal bar size; the purpose of the smaller spacing is to achieve performance capability similar to that of walls with Grade 60 longitudinal reinforcement.

For special structural walls designed with Grade 80 (or Grade 100) main longitudinal reinforcement, ACI Table 18.10.6.5 (b) specifies the following vertical spacing limits:

- Maximum spacing is not more than $5d_b$ or 6 inches for a distance of l_w and $M_u/4V_u$, whichever is greater, above and below critical sections.
- At other locations, maximum spacing shall not exceed $6d_b$ or 6 inches.

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In the above discussion, M_u is the factored bending moment, V_u is the factored shear force, both at the critical section, l_w is the length of the wall and d_b is the diameter of the smallest longitudinal bar in the wall.

Significant revisions other than the one noted above related to Grade 80 reinforcement have also been made to the design and detailing requirements for special structural walls in ACI 18.10 (see CRSI 2020b, c).

TABLE 5.9
Requirements for columns with Grade 80 reinforcement in special moment frames

Requirement	ACI section no.	Figure no.[1]
Transverse reinforcement spacing, s, within a length of at least, l_o. from each end of the column or on both sides of a section where flexural yielding is likely: $s \le$ least of { One – fourth the minimum column dimension; $5d_b$ of the smallest longitudinal bar; $4\text{ in.} \le s_o = 4 + \left[\frac{14 - h_x}{3}\right] \le 6\text{ in.}$ }	18.7.5.3	6
Beyond the length l_o, spacing, s, of transverse reinforcement[2]: $s \le$ least of { 6 in.; $5d_b$ of the smallest longitudinal column bar }	18.7.5.5	6
In accordance with ACI 18.7.5.2 through 18.7.5.4, transverse reinforcement must be provided over the full height of the columns and must be developed into the discontinued member and into the supporting member for columns supporting reactions from discontinued stiff members	18.7.5.6	7

[1] See CRSI 2020c for in-depth discussion on the design and detailing requirements for columns in special moment frames.

[2] The transverse reinforcement shall not be less than that required by ACI 18.7.4.4 or 18.7.6.

[3] h_x is defined as maximum center-to-center spacing of longitudinal bars laterally supported by corners of crossties or hoop legs around the perimeter of a column or wall boundary element, inches.

5.17 DIAPHRAGMS IN BUILDINGS ASSIGNED TO SDC D, E OR F

Mechanical splices used for rebars designed to transfer forces between a diaphragm and the vertical elements of the seismic-force-resisting system (SFRS) have a new

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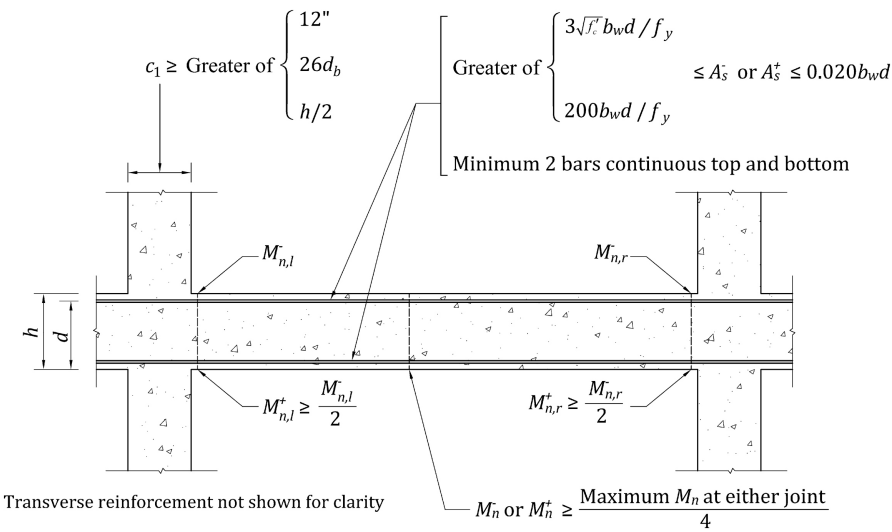
$c_1 \geq$ Greater of 12" / $26d_b$ / $h/2$
Greater of $3\sqrt{f'_c}\,b_w d / f_y$ / $200 b_w d / f_y$
$\leq A_s^-$ or $A_s^+ \leq 0.020 b_w d$
Minimum 2 bars continuous top and bottom
$M_{n,l}^-$
$M_{n,r}^-$
h
d
$M_{n,l}^+ \geq \frac{M_{n,l}^-}{2}$
$M_{n,r}^+ \geq \frac{M_{n,r}^-}{2}$
Transverse reinforcement not shown for clarity
M_n^- or $M_n^+ \geq \frac{\text{Maximum } M_n \text{ at either joint}}{4}$

FIGURE 5.12 Flexural requirements for beams in special moment frames with Grade 80 reinforcement (CRSI 2020b).

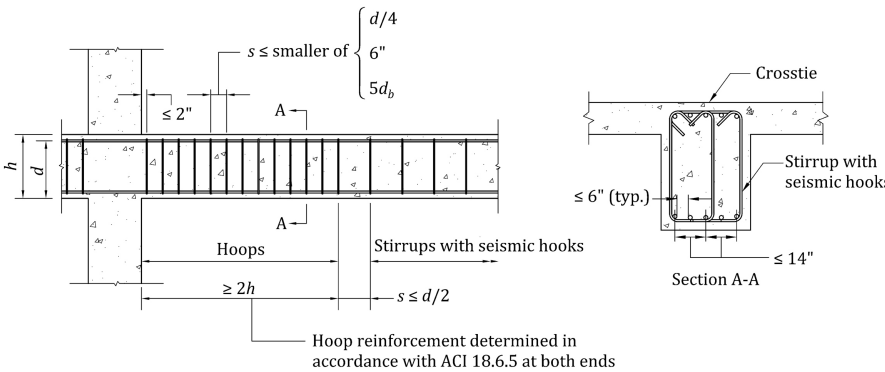

FIGURE 5.13 Transverse reinforcement requirements for beams in special moment frames with Grade 80 reinforcement (CRSI 2020b).

requirement in ACI 18.12.7.4. Grade 80 and Grade 100 reinforcing bars cannot be mechanically spliced for this kind of application.

5.18 LIMITS ON CONCRETE COMPRESSIVE STRENGTH

The revised ACI Table 19.2.1.1 contains limits on f'_c that apply to both regular weight and lightweight concrete. Minimum f'_c is equal to 3,000 psi for special moment frames and special structural walls with Grade 80 longitudinal reinforcing. When

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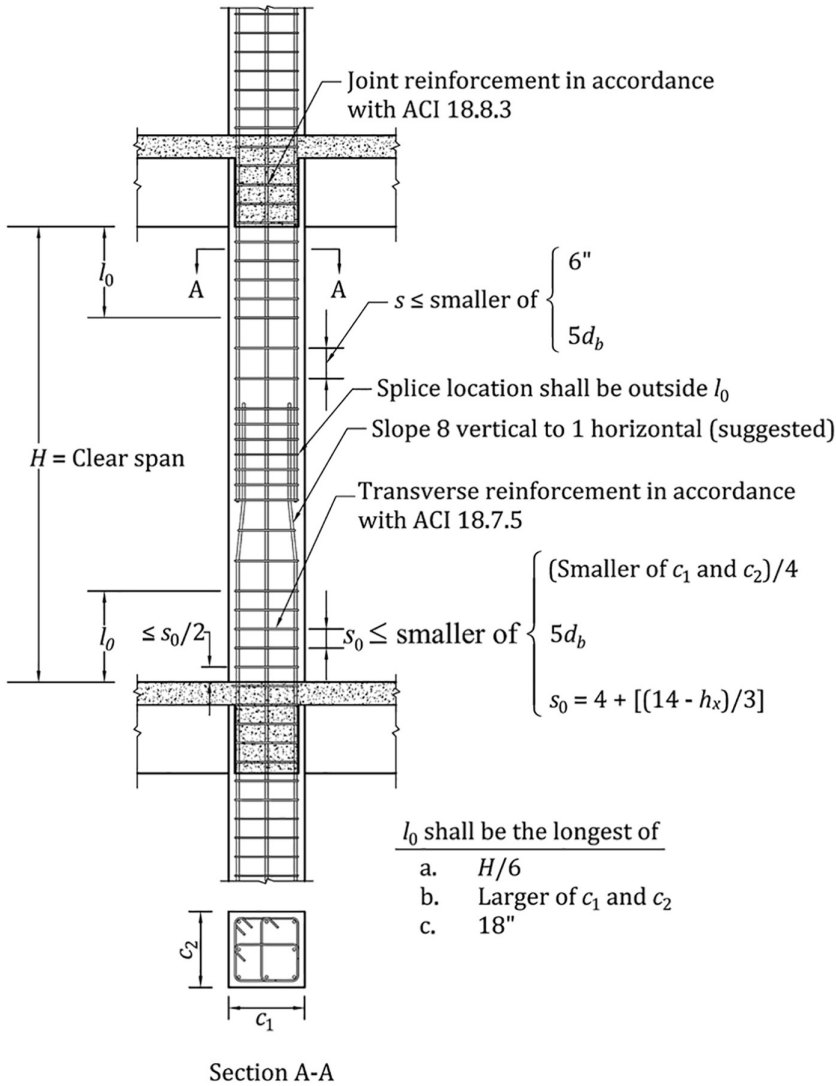

FIGURE 5.14 Transverse reinforcement requirements for columns in special moment frames with Grade 80 reinforcement.

Grade 100 longitudinal reinforcement is used in special structural walls, the minimum value of f_c' is 5,000 psi. These lower limits on f_c' have been imposed because appropriate test data for special moment frames and special structural walls with concrete compressive strengths less than these limits is not available.

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5.19 STRENGTH REDUCTION FACTORS

ACI 21.2.2 introduces a new tension-controlled limit to accommodate for high-strength reinforcement: the net tensile strain in the extreme layer of the main longitudinal tension rebar at nominal strength, ε_t, is greater than or equal to $\varepsilon_{ty} + 0.003$ in tension-controlled concrete sections. In the extreme layer of longitudinal tension rebar used to define a compression-controlled section, the net tensile strain is f_y/E_s, where E_s is the modulus of elasticity of the reinforcing steel. At the extreme concrete compression fiber, the maximum strain is taken as 0.003 (ACI 22.2.2.1).

For sections with Grade 80 reinforcement, the tension-controlled limit is equal to the following as given in Equation 5.5.

$$\varepsilon_t = \left(\frac{80{,}000}{29{,}000{,}000}\right) + 0.003 = 0.0028 + 0.003 = 0.0058 \tag{5.5}$$

In members designed with Grade 100 rebar, the tension-controlled limit is equal to the following as given in Equation 5.6.

$$\varepsilon_t = \left(\frac{100{,}000}{29{,}000{,}000}\right) + 0.003 = 0.0034 + 0.003 = 0.0064 \tag{5.6}$$

Based on revised ACI Table 21.2.2, the variation of the strength reduction factor, ϕ, is given in Figure 5.15.

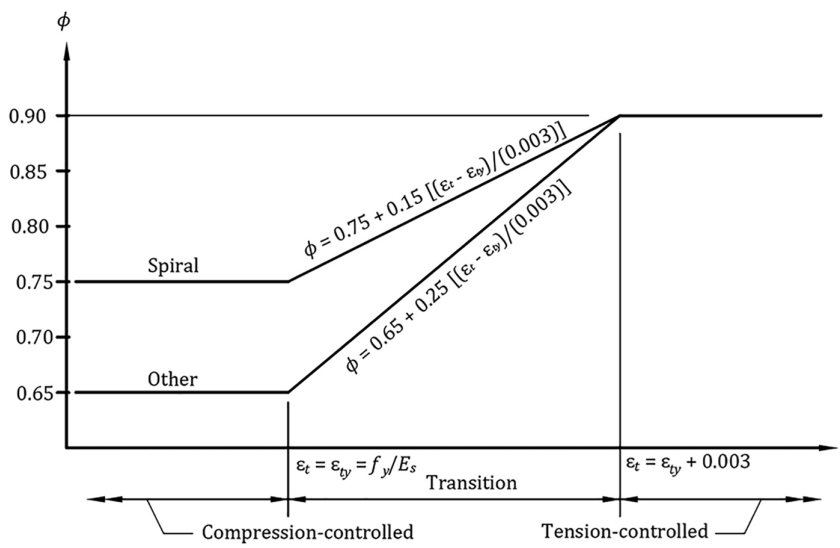

FIGURE 5.15 Variation of strength reduction factor, ϕ, with net tensile strain, ε_t (CRSI 2020b).

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5.20 STRUT-AND-TIE METHOD

The maximum transverse reinforcement spacing in struts with Grade 80 main reinforcement is equal to the lesser of 6 inches and $5d_b$, where d_b is the diameter of the smallest longitudinal bar in the strut (ACI Table 23.11.3.2 (b)).

5.21 NON-LINEAR RESPONSE HISTORY ANALYSIS

The new Appendix A of ACI 318-19 covers standards and parameters for non-linear response history analysis and earthquake-resistant concrete structure design. ACI Table A.9.1 lists the expected material strengths to be considered in the calculations. The predicted yield strength for ASTM A706 Grade 80 reinforcement is 85 ksi, and the expected tensile strength is 112 ksi.

5.22 OTHER DESIGN CONSIDERATIONS

Because development and lap splice lengths are proportional to the yield strength, f_y, these lengths for HSRB are longer, with a percentage increase of 25%, 33%, 67%, and 100% for Grades 75, 80, 100, and 120, respectively, over Grade 60 bars. The use of mechanical splices and headed bars should be considered where appropriate. Headed bar and coupler options are available to anchor and connect HSRB, including large bar sizes; see Chapter 7.

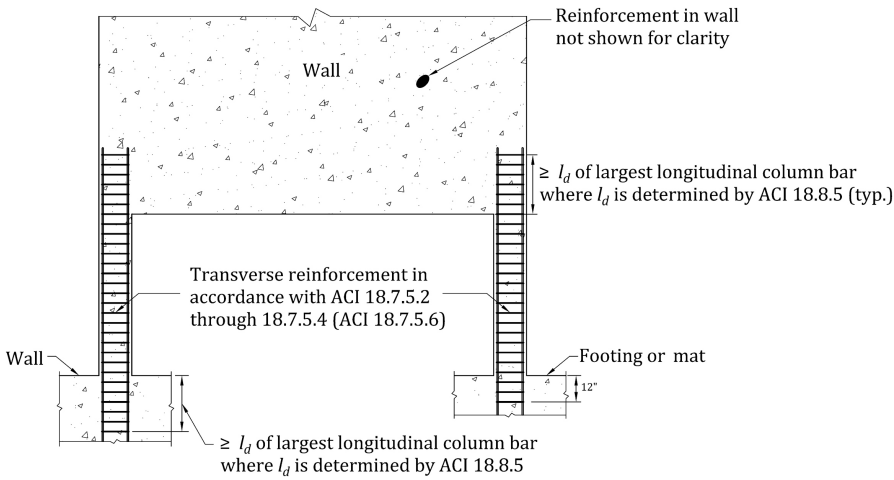
Reinforcement in wall
not shown for clarity
Wall
≥ l_d of largest longitudinal column bar
where l_d is determined by ACI 18.8.5 (typ.)
Transverse reinforcement in
accordance with ACI 18.7.5.2
through 18.7.5.4 (ACI 18.7.5.6)
Wall
Footing or mat
12"
≥ l_d of largest longitudinal column bar
where l_d is determined by ACI 18.8.5

FIGURE 5.16 Transverse reinforcement requirements for columns supporting reactions from discontinued stiff members in special moment frames with Grade 80 reinforcement (CRSI 2020b).

ACI 25.4.4.1 now puts no limits on the yield strength of headed bars. With the inordinately longer development and lap splice lengths of HSRB, a greater emphasis will be placed on utilizing headed bars. Thus, further research is required in order to justify using headed bars to appropriately develop HSRB.

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5.23 SUMMARY

Provisions for the use of higher strength reinforcing bars have been incorporated into ACI 318 as new reinforcing steel products have become available. It is anticipated that this trend will continue as additional research and analytical studies validate the performance of HSRB in reinforced concrete members. Designers can specify and utilize HSRB (with yield strengths greater than 60,000 psi) based on the current provisions and limitations in ACI 318-19. Cost-effective reinforced concrete structures may be realized by taking advantage of the benefits of HSRB, which include smaller required bar sizes and/or a smaller number of required bars, less congestion, lower placement costs, and smaller member sizes.

While there are numerous advantages to employing HSRB, the engineer should exercise caution while selecting it. The author believes that engineers may encounter the following issues during design with HSRB.

- Engineers should continue to utilize f_y of 60 ksi in shear friction calculations. As the total amount of longitudinal reinforcement is decreased by HSRB, shear friction may begin to influence design. If shear friction becomes critical in the design, then roughening construction practices should receive increased consideration.
- Larger crack widths are associated with HSRB yielding. This may negatively impact certain criteria for serviceability and durability, such as steel corrosion. The designer should consider this issue while utilizing HSRB. When serviceability criteria govern the design, the use of HSRB may not be very beneficial.
- All HSRB should be very clearly marked to distinguish it from typical reinforcement on a job site; a common solution is the use of spray paint.
- Diagonal coupling beams, challenging to construct and typically heavily congested, could reduce diagonal reinforcement congestion through the use of HSRB up to Grade 100. A secondary advantage is the construction of a more favorable tie angle in the member, which makes more efficient use of the diagonal reinforcement; this advantage is particularly obvious for shallow diagonal coupling beams.
- Mechanical couplers are not allowed in plastic hinge zones utilizing HSRB; however, the code commentary permits an exception if sufficient product data is presented.
- When the usage of HSRB indirectly lowers the redundancy of reinforcement, caution should be exercised. An example would be reducing the reinforcement of a chord or collector from two bars (total) to one bar, hence reducing its redundancy in the event of a bar defect or splice failure.
- Compression members employing HSRB are capable of attracting and sustaining greater demand. As a result, buckling becomes an important factor to address. Although columns often come to mind in this application, the ends of thin shear walls, particularly those with asymmetric T- or L-shaped arrangements, might also be problematic.

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- Anchorage and force transmission should generally be of greater concern now that higher bar forces are being transmitted. Anchorage or bond failures are more brittle and could prevent a ductile mechanism from functioning as planned. Insufficient tie development within the nodal zones of strut-and-tie models is an example of this.
- When HSRB is used in beams in special moment frames, the dimension of the column must be sufficient to provide adequate embedment of the longitudinal rebar of the beam. This may often govern the column size.
- The engineer should verify the availability of HSRB with vendors. Various bar configurations may have size restrictions imposed by the manufacturer.

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Fanella, D. A. (2011), *Reinforced Concrete Structures Analysis and Design*, McGraw-Hill, New York.

QUESTIONS

5-1. What are the benefits of high-strength reinforcing bars? Discuss.
5-2. How does the stress-strain behavior of Grade 100 rebar affect the flexural and axial load strength of concrete members? Discuss.
5-3. How does the stress-strain behavior of Grade 100 rebar affect the deflection and creaking behavior of concrete members? Discuss.
5-4. Write down the benefits of using ASTM Grade 80 and Grade 100 rebar in structural concrete elements. Mention a few examples.
5-5. In what kind of design applications is the full yield strength of Grade 100 rebar allowed in design calculations?
5-6. How do you define the concept of development length of straight rebar? List the factors on which the development length depends.
5-7. With a neat sketch, describe hoop spacing requirements for columns with Grade 80 main reinforcement in intermediate moment frames.

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5-8. Briefly discuss the applicability of Grade 80 and Grade 100 rebar in special moment frames.
5-9. How is the minimum column dimension in a special moment frame governed by the use of Grade 80 rebar?
5-10. Discuss the hoop spacing requirements in beams of special moment frame when Grade 80 main reinforcement is used.
5-11. With a neat sketch, describe hoop spacing requirements for columns with Grade 80 main reinforcement in special moment frames.
5-12. Describe the issues a design engineer must keep in mind while using high-strength rebar in concrete structures.

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6 Placing and Fabrication Details

6.1 IDENTIFICATION OF BARS

With the different types, grades, and sizes available, it is necessary to provide some means of bar identification. The ASTM specifications require that each bar producer roll the following onto the reinforcing bar (CRSI 2011).

a. The producer's mill is indicated by a letter or symbol.
b. The size number of the bar is indicated by a number or symbol.
c. A symbol or marking to indicate the type of reinforcing steel, and
d. Grade mark, as shown in Figure 6.1, to designate Grades 60 [420], 80 [550], etc.

The grade mark for Grade 60 [420] may be either the number "60" [4] or one (1) single line. When a single line is used to designate Grade 60 [420], it is smaller and between the two main ribs, which are on opposite sides of all reinforcing bars produced with US-style marking. Similarly, the grade marks for Grades 80 [550] are as shown in Figure 6.1.

If grade lines are used, they must be continued with at least five deformation spaces. No grade mark is required for Grades 40 and 50 [280 and 350]. Stainless steel reinforcing bars (ASTM A955) only require the first two identification marks (producer's mill and bar size). Bar identification marks may also be oriented to read horizontally (at 90° to those illustrated in Figure 6.1).

The site personnel should verify these markings to be certain the correct types, grades, and sizes are being placed in the locations where they are intended. If there is any doubt, the ironworker should check with the foreman.

6.2 FABRICATION OF REINFORCING BARS

The fabrication of reinforcing bars includes the cutting-to-length of straight bars, the cutting-to-length and bending to shape of bent bars, and the bundling and tagging of bars for handling and for identification at the job site. These operations in bar fabrication details are the ultimate representation of the engineer's design calculations. Thus proper rebar fabrication details are very important for the successful construction of a concrete structure. A general knowledge of the operations, which occur before delivery of the bars to the job site, may be helpful to the ironworker (CRSI 2011).

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DOI: 10.1201/9781003585657-6

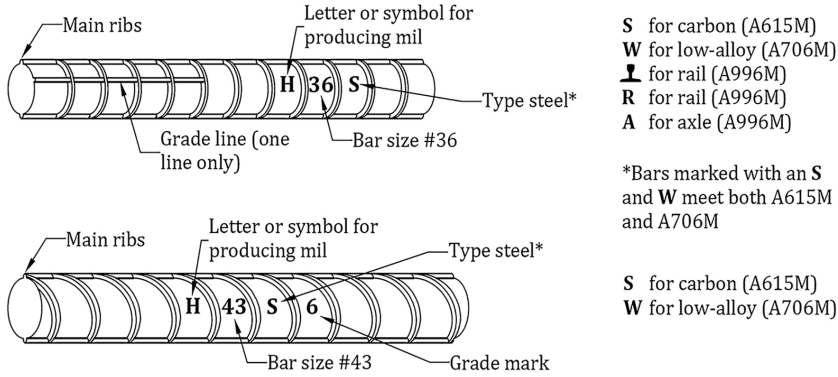
Main ribs
Letter or symbol for producing mil
H 36 S
Type steel*
Grade line (one line only)
Bar size #36
S for carbon (A615M)
W for low-alloy (A706M)
for rail (A996M)
R for rail (A996M)
A for axle (A996M)
*Bars marked with an S and W meet both A615M and A706M
Main ribs
Letter or symbol for producing mil
Type steel*
H 43 S 6
Bar size #43
Grade mark
S for carbon (A615M)
W for low-alloy (A706M)

FIGURE 6.1 Reinforcing bar markings (CRSI 2011).

Straight bars are cut to specified lengths from longer stock lengths in a fabricating shop. All the bars of one size and length are cut and assembled for bundling.

Bent bars are cut to length as described for straight bars and sent to the bending area of the fabricating shop, where they are assigned to the bending machines best suited for the types of bends and sizes of reinforcing bars.

6.2.1 Standard Bar Bends

The most common bend types have been standardized in the ACI 315 standard. Note that additional, non-standard bend types may be required on some projects, and their shape or configuration will vary depending on the project design. Bend type numbers have been given to the various reinforcing bar bend configurations. The ironworker will often find these bend-type numbers shown on placing drawing schedules, bar lists, or bar tags. A reference to a bending type will immediately identify the bend configuration of the bar.

Bend types have a letter assigned to each dimension of the bent bar. When the dimensions are used on a bar list, the bend configuration of the bar shown will be fabricated. By omitting certain letters, a shape may be modified from that shown. For example, by omitting the dimension "G" from Type 1, a bar with only one hook would result. By omitting dimensions "A," "G" and "J" from Type 3, a truss bar without hooks would result. See Figure 6.2 for examples of standard bar bends.

6.2.2 Spirals

Spirals are used in spirally-reinforced columns, piers, and piles (Figure 6.3) and are made of deformed bars, or plain bars, or wire bent to a specified diameter into a shape similar to that of a coiled spring. The spacing of the bar or wire is important and spacers are sometimes provided to hold this spacing (known as the pitch). The spacers also serve to provide sufficient stiffness to the spiral to keep it in alignment while being erected. It is usually a steel channel, or angle punched with a series

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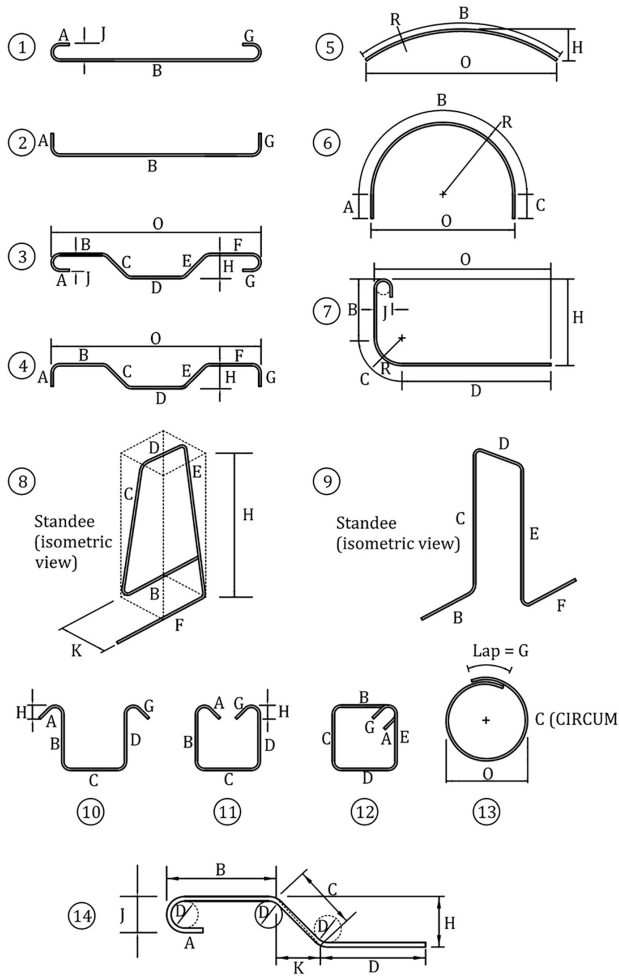

FIGURE 6.2 Examples of standard bar bends recommended by CRSI (2011).

Notes:

a. All dimensions are out-to-out of bar except "A" and "G" on standard 180° and 135° hooks.
b. "J" dimensions on 180° hooks to be shown only where necessary to restrict hook size, otherwise ACI standard hooks are to be used.
c. Where "J" is not shown, "J" will be kept equal to or less than "H" on Bend Types 3, and 5. Where "J" can exceed "H," it should be shown.
d. "H" dimension stirrups to be shown where necessary to fit within concrete.
e. Where bars are to be bent more accurately than standard fabricating tolerances, bending dimensions which require closer fabrications should have limits indicated.
f. For recommended diameter "D," of bends and hooks, see tables in section 6.3.1.
g. Types 10 to 13 apply to bar sizes #3 through #8 [#10 through #25].
h. Unless otherwise noted, diameter "D" is the same for all bends and hooks on a bar (except for Bend Type 7).

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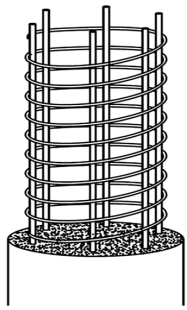

FIGURE 6.3 Spiral in a column (CRSI 2011).

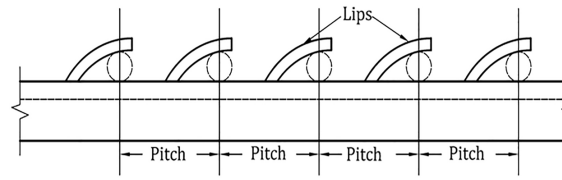

FIGURE 6.4 Typical spiral spacers (CRSI 2011).

of "lips" to hold the spiral turns to the proper pitch, as depicted in Figure 6.4. Each fabricator selects the spacer shape best suited to provide the required stiffness.

TABLE 6.1
Minimum number of spacers in spirals

Spiral bar or wire size	Spiral core diameter, in.	Minimum number of spacers
#3 [#10] or ⅜ in.	Less than 20	2
#4 [#13] or ½ in.	20 to 30	3
	More than 30	4
#5 [#16] or ⅝ in.	24 or less	3
	More than 24	4

TABLE 6.2
Minimum spiral outside diameters

	Minimum spiral outside diameters, in.	
Spiral bar or wire size	Which can be formed	Of collapsible spiral
#3 [#10] or ⅜ in.	8	18
#4 [#13] or ½ in.	10	18
#5 [#16] or ⅝ in.	15	30
#6 [#19] or ¾ in.	24	30

If the engineer has required the use of spiral spacers for the project but has not specified the number of spacers required, Table 6.1 gives the minimum number of spacers required for the size of spiral and the diameter of spiral. Where only two spacers are used, they are placed on opposite sides of the spiral. When more than two spacers are used, the additional ones may be placed adjacent to the two or bundled loosely with the spiral for attachment at the job site. Having the spacers on opposite sides of the spiral permits the spiral to be shipped flat or "collapsed," requiring less space for storage and shipment. At the job site, the spirals are opened, and the spacers shifted, so they are about equally spaced around the perimeter of the spiral.

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Some spirals, because of their small diameter, cannot be collapsed. They are shipped with the spacers mounted and spaced. Table 6.2 shows the minimum diameter of a collapsible spiral related to the spiral size.

Spirals that include spacers are called assembled spirals. In some cases, such as piles, piers, or columns where the pitch is large or where the bar or wire diameter is quite small, or where the spiral can be accurately spaced and tied to longitudinal reinforcing bars, spirals may be furnished into coils to the specified diameter and the proper number of turns without spacers. These are known as unassembled spirals.

6.2.3 Offset Column Bars for Non-Seismic Application

The non-seismic application of offset column bars is shown in Figure 6.5. This detailing is not recommended for seismic application. Offset column bars, as shown in Figure 6.5, are bent so the upper part, which projects beyond the floor above, will fit inside the vertical bars in the column above. The corner bars in a column need to have a larger offset than the bars along the sides, but for simplified fabrication and placing, all offset bends in a column are made using the largest offset. The vertical bending slope should be milder than 6:1 (vertical:horizontal).

Where the column faces are offset 3 inches or more, the lower vertical bars stop below the floor, and a separate short bar called a dowel is used, extending below and above the floor line at a specified distance. In some cases, a column might be offset 3 inches or more on one face only, which could result in some offset bent column bars and some separate dowels being used in the same column.

The most common method used to lap splice column bars is to offset the vertical bars from the floor below (as illustrated in Figure 6.5 (b)). However, there are occasions when it may be beneficial to the project to use the less common method of inverting the offset vertical bars in the column above, as illustrated in Figure 6.5 (d).

6.2.4 Standard Hooks

Standard hooks are used at the end of reinforcing bars when the straight length of the bar cannot provide enough embedment in the concrete to properly anchor the bar. Three types of standard hooks are generally used, known as 90°, 135°, and 180°. The 90° and 180° hooks are standard for all bars except stirrups and column ties, where 90° or 135° hooks are standard. The 90° hook is preferred by most fabricators and ironworkers because of the ease of bending and the ease of placing the bars at the job site. Bars with 90° hooks, when placed in layers, cause less interference with each other. The term “hook” refers to all cases where the dimensions of a bar bent at the ends are designated “A” or “G.” When the designated end dimensions are other than “A” or “G,” the term “bend” is used rather than “hook” (see Figures 6.2, 6.7 and 6.8 and Tables 6.3, 6.4 and 6.5). Dimensions of standard hooks are shown. If a hook is not dimensioned on the project drawings, it will be assumed to be a “standard hook” and detailed to the lengths shown in the schedule.

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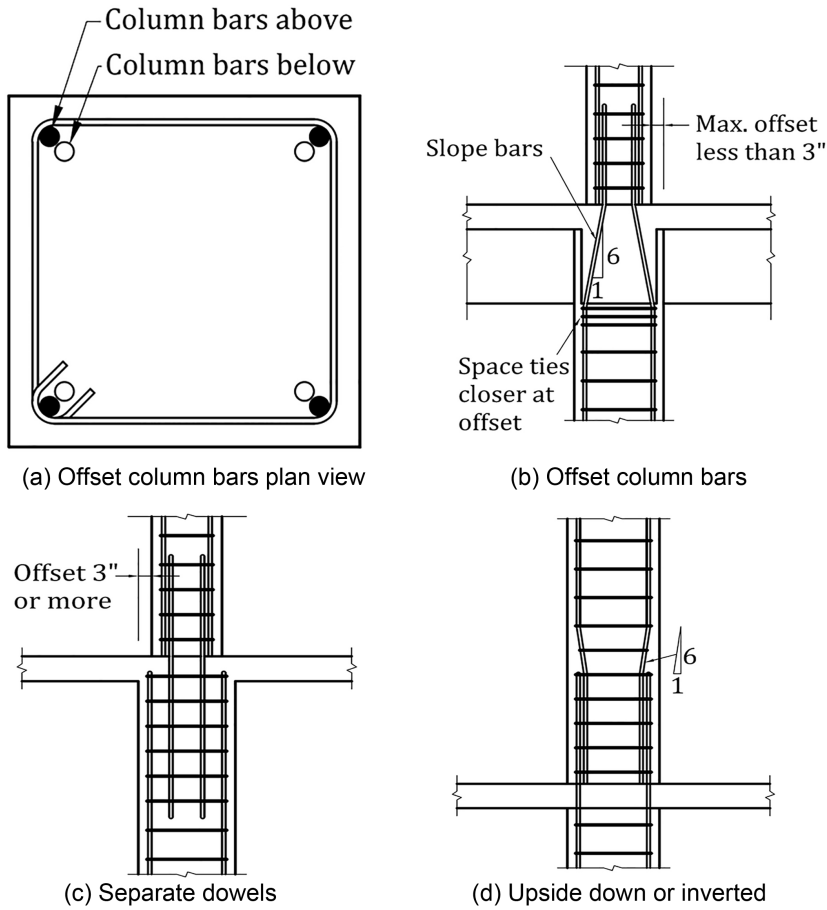

FIGURE 6.5 Offset column bars for non-seismic application (CRSI 2011).

6.3 BAR SUPPORTS

6.3.1 General

Bar supports are intended to maintain the location of the reinforcing bars during the concrete placement. They also ensure proper concrete cover is provided over the reinforcement, and proper design depth is achieved. The concrete cover is very important in concrete construction because it provides a minimum outer dimension of concrete for corrosion protection and fireproofing for the embedded reinforcing steel.

The height of bar support is taken as the distance from the bottom of the leg, sand plate, or runner wire to the underside of the reinforcing bar when the bar is properly positioned on the support. A variation of plus or minus ⅛ inch from the specified nominal height is a normal construction tolerance.

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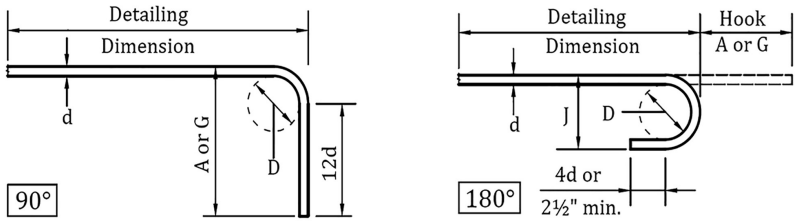

FIGURE 6.6 Standard hooks.

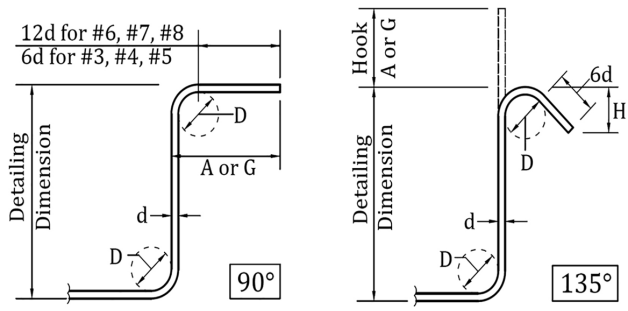

FIGURE 6.7 90° and 135° stirrup/tie hooks.

TABLE 6.3
Recommended end hook dimensions for standard hooks

		180° hooks, (ft.–in.)		90° hooks, (ft.–in.)
Bar size	**D (in.)**	**A or G**	**J**	**A or G**
#3 [#10]	2¼	0–5	0–3	0–6
#4 [#13]	3	0–6	0–4	0–8
#5 [#16]	3¾	0–7	0–5	0–10
#6 [#19]	4½	0–8	0–6	1–0
#7 [#22]	5¼	0–10	0–7	1–2
#8 [#25]	6	0–11	0–8	1–4
#9 [#29]	9½	1–3	0–11¾	1–7
#10 [#32]	10¾	1–5	1–1¼	1–10
#11 [#36]	12	1–7	1–2¾	2–0
#14 [#43]	18¼	2–3	1–9¾	2–7
#18 [#57]	24	3–0	2–4½	3–5

All grades of steel; D = Finished inside bend diameter (includes spring back).

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TABLE 6.4
Recommended dimensions for 90° and 135° stirrup/tie hooks

		90° hooks, (ft.–in.)	135° hooks, (in.)	
Bar size	**D (in.)**	**A or G**	**A or G**	**H (Approx.)**
#3 [#10]	1½	0–4	4	2½
#4 [#13]	2	0–4½	4½	3
#5 [#16]	2½	0–6	5½	3¾
#6 [#19]	4½	1–0	8	4½
#7 [#22]	5¼	1–2	9	5¼
#8 [#25]	6	1–4	10½	6

All grades of steel; D = Finished inside bend diameter.

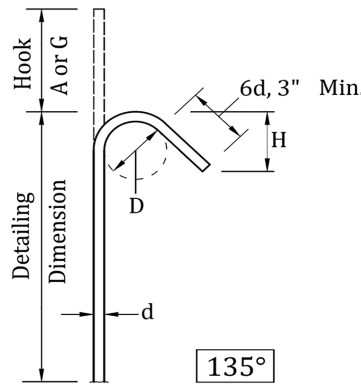

FIGURE 6.8 135° seismic stirrup/tie hooks.

TABLE 6.5
Recommended seismic stirrup/tie hook dimensions

		135° hooks, (in.)	
Bar size	**D (in.)**	**A or G**	**H (Approx.)**
#3 [#10]	1½	4¼	3
#4 [#13]	2	4½	3
#5 [#16]	2½	5½	3¾
#6 [#19]	4½	8	4½
#7 [#22]	5¼	9	5¼
#8 [#25]	6	10½	6

All grades of steel; D = Finished inside bend diameter.

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Bar supports are intended to support the reinforcing steel and normal construction loads and to main the alignment of the reinforcing steel. Bar supports are not intended to support hoses for concrete pumps or runways for concrete buggies or similar loads.

Bar supports may be made of steel wire, precast concrete, or plastic. Illustrations of wire bar support, precast concrete blocks, and all-plastic bar supports are presented in Tables 6.6 and 6.7.

6.3.2 Wire Bar Supports

Wire bar supports are classified in terms of their ability to protect the concrete surface on which they rest from visible rust spots or similar blemishes directly caused by their use. The four classes and their intended degree of corrosion protection are discussed below.

6.3.2.1 Class 1 – Maximum Protection

Plastic protected wire bar supports: these are intended for use in situations of moderate to severe exposure and/or situations requiring light grinding (1/16 inch maximum) or sandblasting of the concrete surface.

6.3.2.2 Class 1a – Maximum Protection (For Use with Epoxy-Coated Reinforcing Bars)

Epoxy-, vinyl-, or plastic-coated bright basic wire bar supports: these are intended for use in situations of moderate to maximum exposure where no grinding or sandblasting of the concrete surface is required. They are generally used when epoxy-coated reinforcing bars are required.

6.3.2.3 Class 2 – Moderate Protection

Stainless steel protected wire bar supports: these are intended for use in situations of moderate exposure and/or situations requiring light grinding (1/16 inch maximum) or sandblasting of the concrete surface. Class 2 protection may be obtained by use of either type of stainless-steel protected wire bar support. The difference between them is the length of the stainless-steel tip attached to the bright basic wire at the bottom of each leg,

Type A – ¼ inch

Type B – ¾ inch

6.3.2.4 Class 3 – No Protection

Bright basic wire bar supports: these have no protection against corrosion and are intended for use in situations where surface blemishes can be tolerated or where the supports do not come in contact with the exposed concrete surface. Sometimes, different classes of wire bar supports are used on the same construction project, so the class for each part of the project must be used as directed on the placing drawings.

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TABLE 6.6
Typical examples of wire bar support (CRSI 2011)

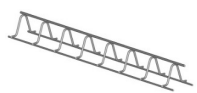
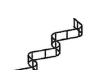
Symbol	Bar support illustration	Bar support illustration plastic tipped	Type of support	Typical sizes
SB, BB, or CHC			Slab bolster or beam bolster or continuous high chair	¾, 1, 1½, 2, and 3 in. heights in 5 ft and 10 ft lengths
SBU, BBU, or CHCU			Slab bolster upper or beam bolster upper or continuous high chair upper	Up to 15 in. high
BC, HC, or HCM			Individual bar chair or individual high chair	¾, to 15 in. heights
JC			Joist chair	4, 5, and 6 in. widths and ¾, 1 and 1½ in. heights
JCU			Joist chair upper	14 in. span; heights −1 in. thru + 3½ in. vary in ¼ in. increments
CS			Continuous support	1½ to 12 in. in increments of ¼ in. in lengths of 6–8 in.

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TABLE 6.7
Typical concrete bar supports

Bar support illustration	Type of support	Typical sizes, mm	Description
	Plain block	20 × 25 × 25 to 40 × 50 × 50	Used when placing bars off grade and formwork.
	Wired block	20 × 25 × 25 to 40 × 50 × 50	Generally, block is cast with embedded 16-gauge tie wire, commonly used against vertical forms or in positions necessary to secure the block by tying to the reinforcing bars.
	Cylindrical block	ϕ 75 × 75 to ϕ 75 × 125	Used to support top mat.
	Side-form spacer	Concrete cover, 50 to 100	Used to align the reinforcing bar cage in a drilled shaft. Supports for 3 in. to 6 in. cover.

6.4 GENERAL PRINCIPLES FOR PLACING, SPLICING, AND TYING REINFORCING BARS

6.4.1 General

Reinforcing bars must be carefully and accurately placed to conform to the placing drawings and requirements of the contract documents. Often the engineer's structural drawings call for certain work to conform to a standard typical detail or note. The instructions called for in such details or notes are shown by the detailer on the placing drawings. Placing drawings should clearly identify such items as top and

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bottom reinforcing bars, bars hooked around other bars, and on which side or face of the concrete member the bar is to be located. Bars must be fitted around sleeves, inserts, holes, and openings as shown on the placing drawings.

The ironworker foreman and inspector should check the structural drawings for typical details and notes to be sure the detailer has accounted for all details on the placing drawings. They should also check notes calling for "Conformity with CRSI Manual of Standard Practice" or the "ACI Detailing Manual."

Before starting the project, the ironworker foreman should discuss with the inspector and the engineer all aspects of the project to be sure there is a clear understanding of the requirements. In some cases, it may be necessary to talk to the fabricator who is responsible for preparing the placing drawings.

6.4.2 Concrete Cover or Protection

The proper amount of concrete cover on reinforcing bars is very important. The engineer specifies the amount of concrete protection for each part of the structure and bases the design on these cover requirements, taking into consideration the requirements of building codes, fire hazards, the possibility of corrosion, and exposure to weather.

Where not specified, the following minimum concrete cover (outside of the bar to the face of concrete) should be observed:

a. 3 inches at sides where concrete is cast against the ground and on the bottoms of footings or other principal structural members where concrete is placed on the ground (Table 6.8 (f)).
b. 2 inches for bars larger than #5 [#16], where concrete surfaces, after removal of forms, will be exposed to the weather or be in contact with the ground, and 1½ inches for #5 [#16] bars and smaller.
c. 1½ inches over ties and spirals in columns (Table 6.8 (a)).
d. 1½ inches to the nearest reinforcing bars on the top, bottom, and sides of beams and girders (Table 6.8 (b)).
e. ¾ inch for #11 [#36] and smaller bars on top, bottom, and sides of joists, and on top and bottom of slabs where concrete surfaces are not exposed directly to the ground or the weather and 1½ inches for #14 and #18 [#43 and #57] bars (Table 6.8 (c)).
f. ¾ inch from the faces of all walls not exposed directly to the ground or the weather for #11 [#36] and smaller bars, and 1½ inches for #14 and #18 [#43 and #57] bars (Table 6.8 (d)).

6.5 TYPICAL REINFORCEMENT DETAILS

Following the detailing provisions of the ACI and BNBC codes, some typical rebar detailing that may be followed in concrete construction are given here. These detailing are generally applicable for most cases of concrete building construction, including general seismic applications.

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6.5.1 Lap Splice Location in Beams

Lap splice location in beams shall be in accordance with the provision shown in Figure 6.9 for seismic applications. No more than 50% of the longitudinal bars shall be spliced at one place of the beam. Lap splices are to be confined by hoops/stirrups with a maximum spacing of 100 mm.

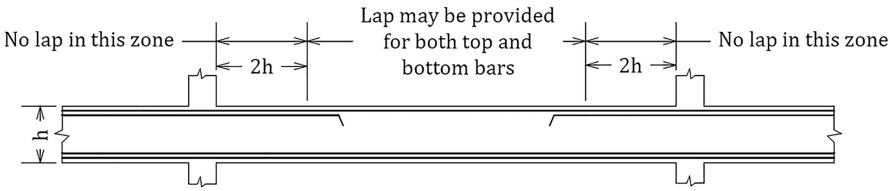

FIGURE 6.9 Suggested lap splice location in typical beams in seismic applications.

6.5.2 Lap Splice Location in Columns

For seismic applications, lap splices must be avoided near the floor or near the beam-column junction. All lap splices in columns shall be provided within the middle half height of the column, as shown in Figure 6.10. Further details on lap splice location in columns can be found in Secs. 5.13 and 5.14.1. Lap splices are to be confined by hoops/ties with a maximum spacing of 125 mm.

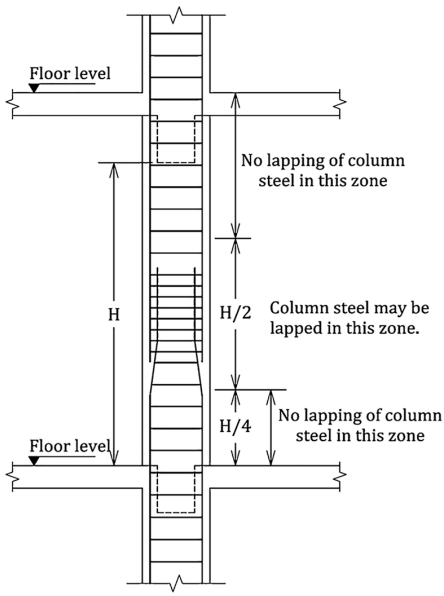

FIGURE 6.10 Lap location details for columns in seismic application.

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6.5.3 Column Tie Details

Ties shall be arranged such that corner hooks grab the main reinforcement in an alternate fashion (Figure 6.11). Tie spacing shall be in accordance with those shown in Figures 5.11 or 5.14. Hooks of column ties shall be bent 45° inwards, and the length of the hook shall be a minimum 75 mm as shown in Figure 6.12 and shall also conform to Figure 6.8.

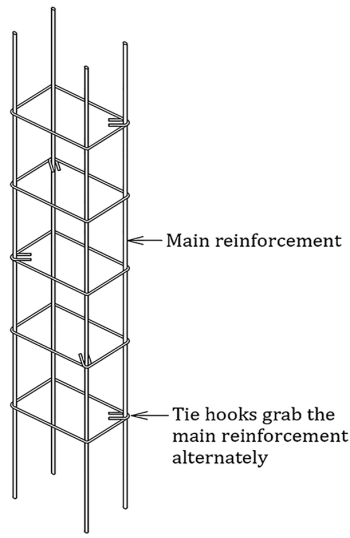

FIGURE 6.11 Vertical arrangement of tie hooks.

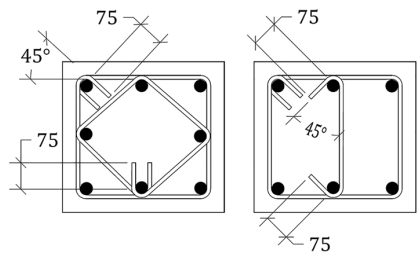

FIGURE 6.12 Geometric details of column tie hooks.

6.5.4 Beam Stirrup

Hooks in beam stirrups shall be bent 45° inwards, and the length of the hook shall be a minimum 3 in. (75 mm), as shown in Figure 6.13. Stirrups shall be arranged such that corner hooks grab the main reinforcement in an alternate fashion.

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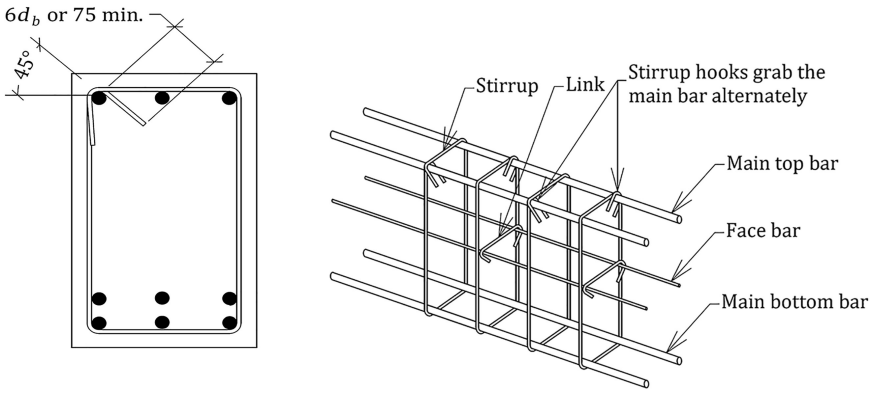

FIGURE 6.13 Beam stirrup details.

6.5.5 End Anchorage for Horizontal Reinforcement (in Beam, Slab, Mat, etc.)

90° hooks as shown in Figure 6.14 shall be provided for a) all main bars of beams (top and bottom) which terminate into columns or walls or other beams, b) slab top bars terminating into supporting beams or walls, c) mat reinforcement terminating at the periphery. For all cases, the length of the hook shall be 12 times the bar diameter ($12d_b$).

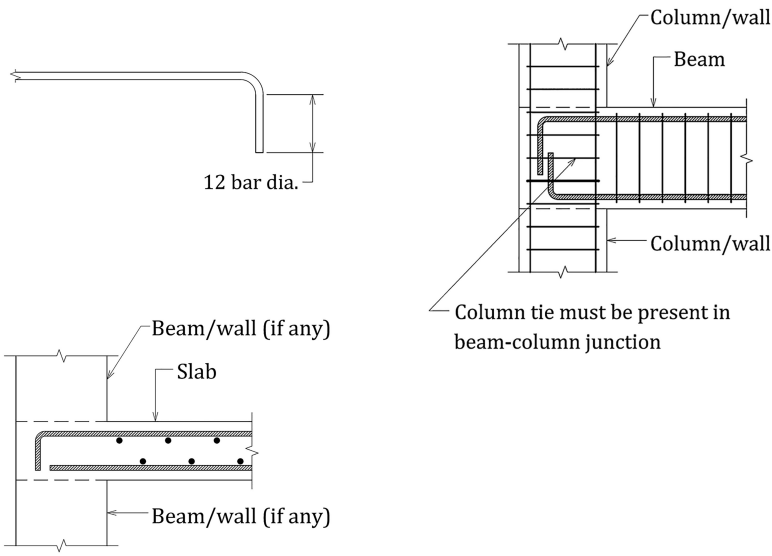

FIGURE 6.14 End-anchorage or 90° hook for main rebar.

6.5.6 Minimum Clear Cover for Reinforcing Bars

Under normal exposure conditions, the minimum concrete cover for reinforcing bars shall be as shown in Table 6.8. In all cases, the cover is measured from the concrete

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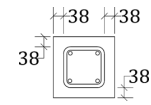
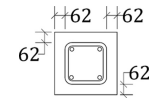
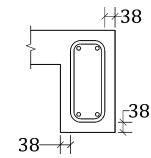
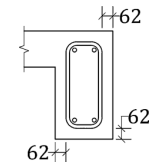
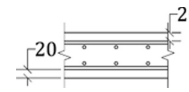
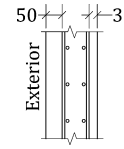
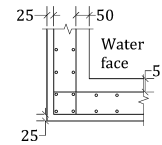
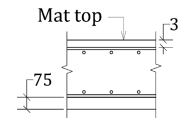
surface to the nearest reinforcement surface e.g., the surface of ties in the column, stirrups in beams, etc.

TABLE 6.8
Clear cover of concrete elements

Sl	Member	Condition	Thickness of cover	Figure
a.	Column, lift core, and shear wall	Above ground level	38 mm	
		Below or in contact with ground	63 mm	
b.	Beam	Top, side, and bottom	38 mm	
		Water face inside water tank: side and bottom	63 mm	
c.	Slab and stair	Top and bottom	20 mm	
d.	R.C.C. wall below ground	Exterior Interior	50 mm 38 mm	
e.	Water tank	Water face Other face	50 mm 25 mm	
f.	Footing and mat foundation	Top Bottom	38 mm 75 mm	

6.5.7 Typical Mat Reinforcement Details around Dead Pile

Suggested details of integrating king posts or dead-piles with mat foundation are shown in Figure 6.15.

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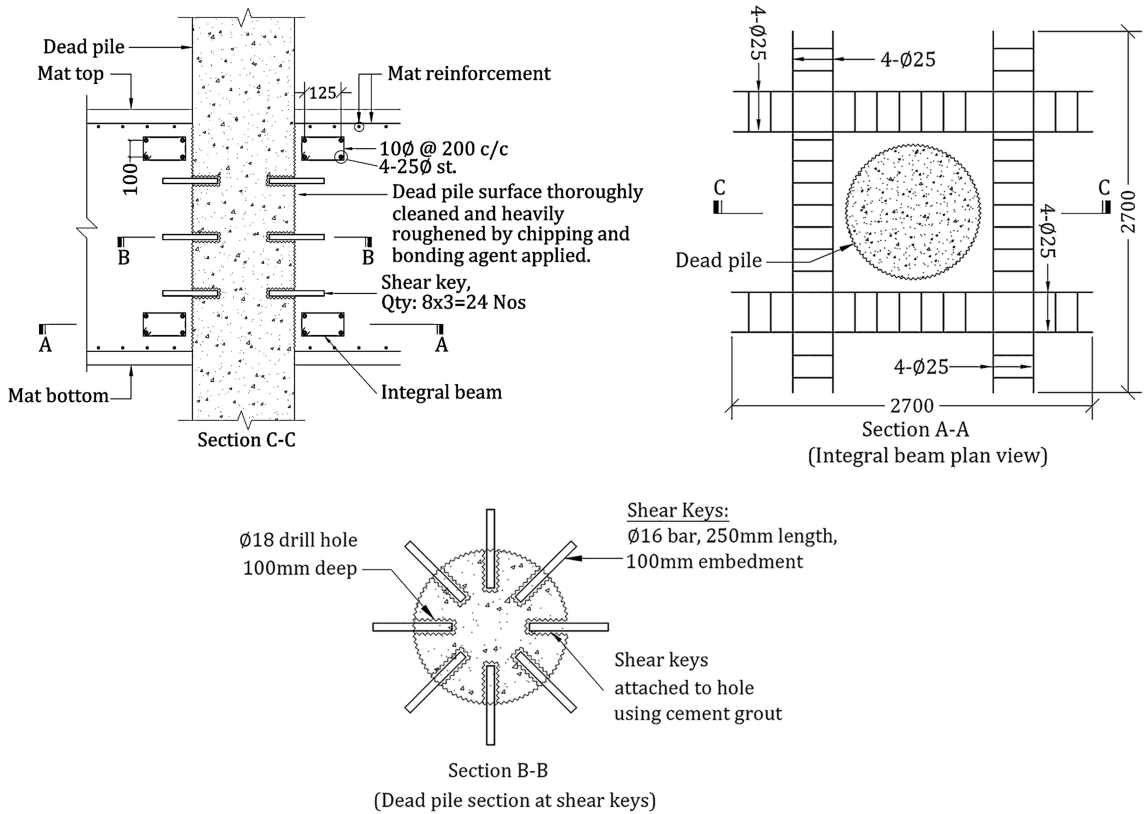

FIGURE 6.15 Suggested detail for integrating dead pile with mat foundation.

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6.5.8 Avoiding Conflict between Beam and Column Reinforcement

If a conflict arises between beam and column or wall reinforcement when beam steel enters or passes through the column, the beam reinforcement may be horizontally bent inwards into the column, as shown in Figure 6.16.

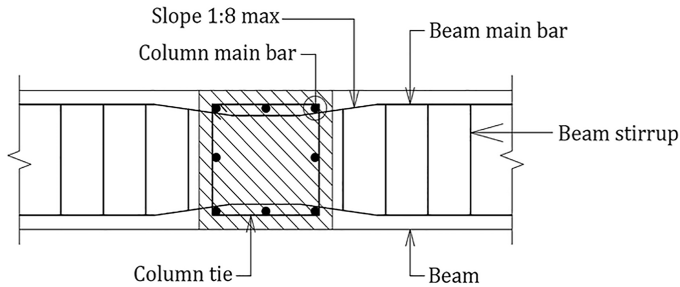

FIGURE 6.16 Suggested detail to avoid conflict between column and beam rebar.

6.5.9 Reinforcement Detailing around Void/Opening

Suggested reinforcement details around void/opening in floor slabs is shown in Figure 6.17. The detailing is generally valid for a maximum void size of 900 mm × 900 mm and when the live load is not more than 5.0 kN/m^2. For void/opening of larger size or for higher live load, reinforcement detailing shall be on the basis of a rational analysis.

	Void/opening size less than 450	Void/opening size between 450~900
Dim. a	900	1500
Dim. b	600	900
Bar size	Ø16	Ø20

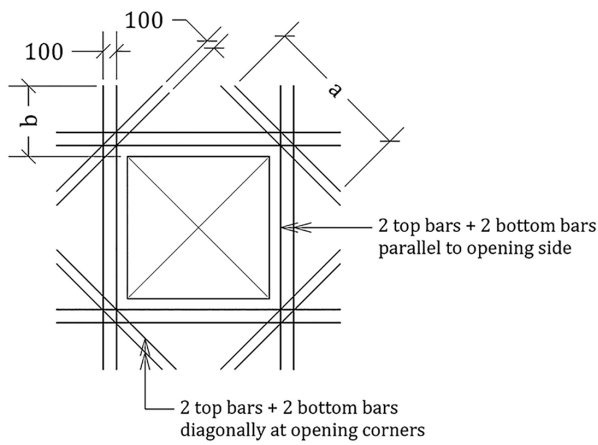

FIGURE 6.17 Suggested rebar detail around a void in slab.

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6.5.10 Column Offset Detail

When column size is reduced, the transition of column main reinforcement from lower floor to the upper floor may be detailed, as shown in Figure 6.18.

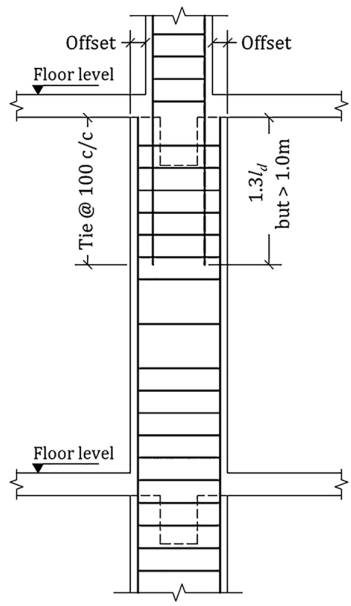

FIGURE 6.18 Column offset detail.

6.5.11 End Anchorage of Beam Reinforcement

Minimum end anchorage length l_{dh} as shown in Figure 6.19 for different bar sizes must be maintained throughout. The value of l_{dh} shall be in accordance with sec. 5.9.2.

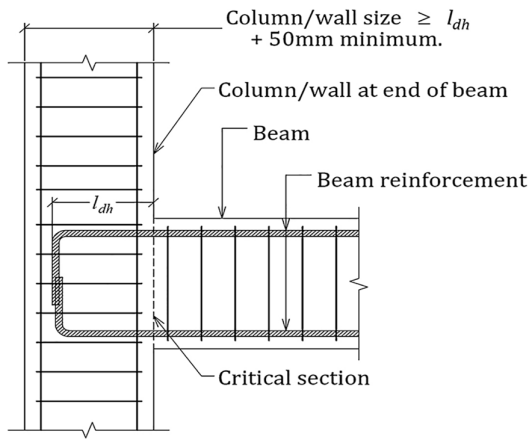

FIGURE 6.19 90° bent of beam rebar in a column.

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REFERENCES

ACI 315R-18 (2018), *Guide to Presenting Reinforcing Steel Design Details*, American Concrete Institute, Farmington Hills.

CRSI (2011), *Placing Reinforcing Bars*, 9th Edition, Concrete Reinforcing Steel Institute, Schaumburg, IL.

QUESTIONS

6-1. What is meant by bar identification? Please describe the requirements for bar identification according to ASTM specifications.

6-2. Why are proper rebar fabrication details important? Explain.

6-3. With neat sketches, discuss the detailing method for offset column bars in non-seismic applications.

6-4. Discuss concrete clear cover requirements for beams, slabs, columns, and walls. What type of bar supports are used to maintain these covers? Illustrate with sketches.

6-5. With neat sketches, show the following reinforcement details:
   a. Avoiding conflict between beam and column reinforcement.
   b. Reinforcement detailing around void opening.
   c. End anchorage for horizontal reinforcement in beams and slabs.
   d. Lap location details for columns in seismic applications.

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7 Rebar Splicing and Coupling

7.1 GENERAL

Most of the time, it is not possible to lay a single piece of rebar from one end to the other of a building or structure. This is so because rebar is rolled out from the mill in certain lengths. Most commonly, the length is about 12 m to 15 m. This is generally governed by the length of available truck trailers on which the rebar is transported. For this reason, rebar is frequently spliced or joined at the site to provide *structural continuity* of reinforcement in a concrete structural element. Splicing the rebar in a proper way during the construction is extremely important as it ensures the proper transfer of bar force from one bar to its counterpart in the spliced connection. There are several different ways to splice rebar; the most common methods are:

a. Lap splice,
b. Mechanical splice, and
c. Welded splice.

The following sections discuss the above three types of splices in detail.

7.2 LAP SPLICING

Lap splices are the most common sort of splice and are usually cost-effective. The bars are normally in contact over a certain length and are tied together in a lap splice as shown in Figure 7.1. This is generally termed as contact lap splice. The length of the lap depends on the strength of concrete and rebar as well as many other factors. Bond transfers the force from one bar to the surrounding concrete, which then transfers it to the adjacent bar. If the concrete surrounding the lap splice is cracked or damaged, it will fail to transfer the force from one bar to the counterpart bar, and the splicing will become ineffective. Obviously, the integrity of the concrete surrounding the splice is extremely important. Laps should be provided at locations where cracks and damage to the concrete are less likely to occur due to loading. In some instances, code also specifies that confining reinforcement such as ties or stirrups be provided at the lap locations. The following should be considered when adopting lap splices:

- Splices shall be located away from points of maximum flexural stress.
- Splice location shall be staggered wherever possible.

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DOI: 10.1201/9781003585657-7

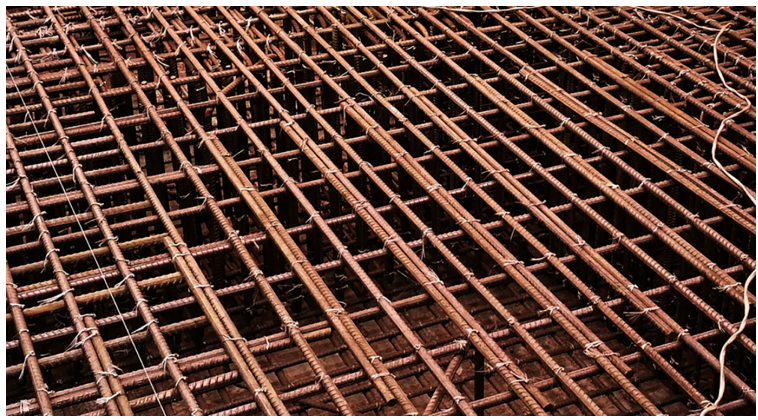

FIGURE 7.1 Lap splicing of rebar.

The length of the lap splice is determined by the bars' tension development length, the area of reinforcement used over the length of the splice, and the proportion of reinforcement spliced at the position. Details of the recent provisions on tension development length, l_d, are discussed in Section 5.9. Because experimental data on lap splices using bars larger than #11 are sparse, the use of tension lap splices is prohibited (ACI 25.5.1.1) for bar sizes greater than the #11 bar.

The new requirement in ACI 25.5.1.5 pertaining to confinement of lap splices is the same requirement in ACI 25.4.2.2 for the confinement of deformed bars in tension: for lap-spliced bars with $f_y \geq 80{,}000$ psi and a center-to-center spacing less than 6 inches, transverse reinforcement must be provided along the entire lap splice length such that $K_{tr} \geq 0.5d_b$ (see Section 5.9.1 for the definition of K_{tr}). Some amount of transverse reinforcement is essential to confine Grade 80 and Grade 100 bars to prevent splitting failures due to relatively large axial forces in the bars.

7.2.1 Compression Lap Splice

Requirements for compression lap splice lengths have been modified to account for higher strength reinforcement. For Grade 80 reinforcement, the compression lap splice length, l_{sc}, is equal to the following (ACI 25.5.5.1) as given in Equation 7.1.

$$l_{sc} = \text{longer of} \begin{cases} \left(0.0009 f_y - 24\right) d_b \\ 12 \text{ in.} \end{cases} \tag{7.1}$$

For Grade 100 reinforcement, the compression lap splice length, l_{sc}, is equal to the following (ACI 25.5.5.1) as given in Equation 7.2.

$$l_{sc} = \text{longer of} \begin{cases} \left(0.0009 f_y - 24\right) d_b \\ l_{st} \end{cases} \tag{7.2}$$

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where l_{st} is the tension lap splice length calculated in accordance with ACI 25.5.2.1 (which has not been revised).

Tests have shown that splice strengths in compression depend mostly on end bearing and do not increase proportionally in strength when the lap splice length is doubled.

7.2.2 Tension Lap Splice

Tension lap splices are classified as Class A or Class B, depending on the lap length. The lap splice length, l_{st}, is given as a multiple of the tension development length l_d:

- Class A splice length = $1.0l_d \geq 12$ in.
- Class B splice length = $1.3l_d \geq 12$ in.

ACI stipulates that l_d be determined in conformance with the provisions of ACI 25.4.2.4; however, a 12-inch minimum length is specified in ACI 25.4.2.4. The modification factors regarding excess reinforcement of ACI 25.4.10.1 are generally not applicable in most cases (the splice length must be based on the full f_y, because the provided area of reinforcement is taken into account in the definition of the splice classification). The effective clear spacing is illustrated in Figure 7.2 for a beam that is to be used in the calculation of l_d. In one-way slabs, the minimum distance between adjacent splices shall be taken as the clear distance between staggered splices (see ACI Figure R25.5.2.1) .

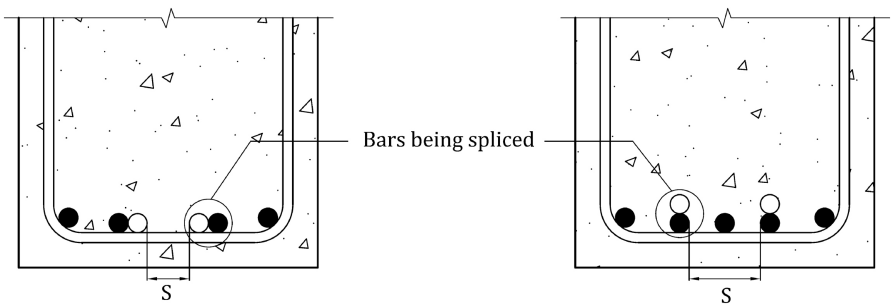

FIGURE 7.2 Clear spacing of spliced bars in a beam (Fanella 2011).

Unless otherwise specified, the default splice classification for a lap splice is Class B. However, Class A lap splice is permitted if both of the following two conditions are satisfied:

- The provided $A_s \geq 2$ times the required A_s over the entire splice length.
- No more than 50% of the rebar is spliced within the required lap length.

The first of these two requirements essentially encourages splices to be located away from high tensile stress areas in the bars. In the absence of load reversals (e.g., due

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to seismic or wind effect), negative reinforcing bars should be spliced toward the midspan of a member, while positive reinforcing bars can be spliced near the supports. For seismic applications, the provision outlined in Figure 6.9 shall be applied.

Tests have demonstrated that transverse reinforcement slows or eliminates the initiation of splitting cracks at the ends of a splice. Therefore, transverse reinforcing, such as stirrups, should be used over the length of a lap splice to increase its capacity. Utilizing a specific amount of transverse reinforcement can also reduce splice length (see ACI 25.4.2.3).

When bars of different sizes are lap spliced in tension at the same location, ACI 25.5.2.2 provides the requirements. For such cases, the minimum lap splice length is the larger of the following: (1) tension development length, l_d, of the larger diameter bar, and (2) tension lap splice length of the smaller diameter bar.

In flexural members, non-contact lap splices are allowed. In contrast to contact lap splices, the spliced bars are not in contact in such as splice. According to ACI 25.5.1.3, the distance between the bars cannot exceed the lesser of (1) ⅕ the required lap splice length or (2) 6 inches. Any distance greater than the limiting distance can result in a portion of concrete between the bars to be unreinforced. Contact lap splices are typically preferred over non-contact splices because the risk of bar displacement during concrete application is substantially lower with contact splices.

7.3 MECHANICAL SPLICING OR COUPLING

An alternative to lap splicing is mechanical splicing where two bars are mechanically connected to each other by means of a coupling device that structurally joins them. The connection is such that the force from one side is transferred to the other side through the coupling system and without the help of the bond of rebar with concrete. In ACI 439.3, a mechanical splice is defined as a "*complete assembly of a coupler, a coupling sleeve, or an end-bearing sleeve, including any additional intervening material or other components required to accomplish the splicing of reinforcing bars*." Different features of mechanical splice are discussed below.

a. **Mechanical splice provides structural continuity in the rebar.** Lap splices rely on the rebar/concrete bond and concrete integrity to transfer the load from one rebar to the other. Any disruption or damage to the concrete around rebar will cause a reduction in the capacity of the lap splice. On the other hand, mechanical couplers ensure structural continuity of the rebar without relying on concrete bond or concrete integrity to transfer the load (Figure 7.3). For this reason, ACI specification allows use of couplers (Type 2) with Grade 60 rebar within the column near the beam-to-column joint in special seismic application but does not allow any lap splicing as an alternate in such applications.
b. **Mechanical splice reduces material usage.** The use of mechanical couplers allows for the effective use of material and a reduction in the quantity of steel used. Table 7.1 shows a comparison between the additional weight of rebar required in a traditional lap splice with the corresponding weight

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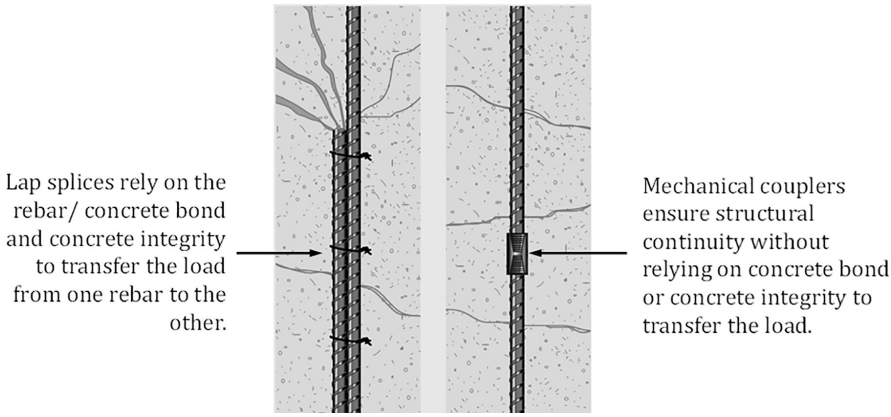
Lap splices rely on the rebar/ concrete bond and concrete integrity to transfer the load from one rebar to the other.
Mechanical couplers ensure structural continuity without relying on concrete bond or concrete integrity to transfer the load.

FIGURE 7.3 Lap splice vs. mechanical splice in transferring force in the rebar.

TABLE 7.1
Rebar weight in lap-splice vs. coupler weight

Class B lap splice length for deformed MS rebar for f'_c = 35 MPa and f_y = 415 MPa (ACI 318-19).

	Tension rebar				
Bar diameter	Top bars in beam and mat; top bars in slab of thickness more than 300 mm	Bottom bars in slab, beam, and mat; top bar in slab of thickness less than 300 mm	Vertical compression rebar in column and shear wall	Weight of one rebar in lapped length	Max. coupler weight
mm	mm	mm	mm	kg	kg
20	1150	875	600	1.50~2.84	1.00
25	1800	1375	750	2.89~6.94	1.50
28	2000	1550	850	4.10~9.67	2.00
32	2300	1775	950	6.00~14.52	3.00
40	2875	2200	1200	11.84~28.36	3.50

of a mechanical coupler assembly. It can be observed that significant savings in the material can be obtained specifically for larger diameter bars if a mechanical coupling system is used instead of lap splicing.

c. **Mechanical splice reduces rebar congestion.** One of the main reasons for the formation of rock pockets and voids in the concrete is lap splicing, which increases rebar congestion at the lap zone. Using mechanical splices can reduce these congestion issues and increase overall job efficiency, specifically at beam-column joints where rebar from three directions cross the same zone leading to congestions (Figure 7.4). Using mechanical splices

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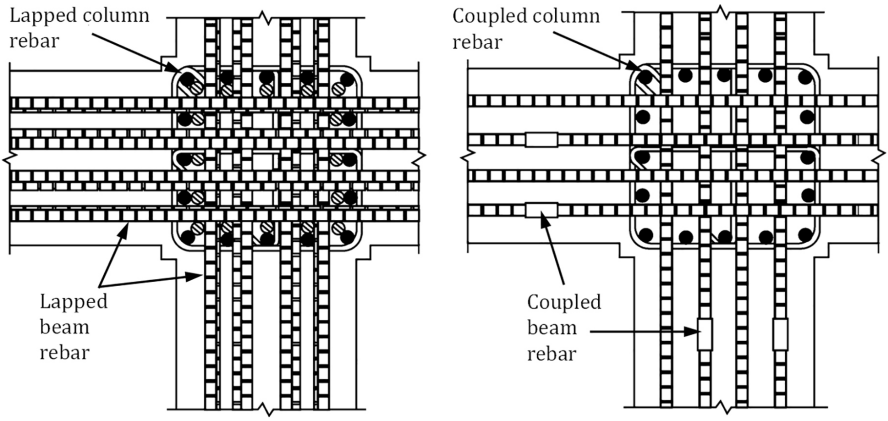

FIGURE 7.4 Lapped vs. mechanically spliced rebar in a beam-column joint (plan views).

allows for rebar with a bigger diameter to be used in a smaller column while minimizing congestion.

d. **Other features.** When mechanical couplers are used, prefabricated rebar cages can be easily connected on site, making on-site welding of rebar unnecessary. Individual concrete elements and construction joints can be joined without the need to drill through shuttering. The use of couplers allows for the face of concrete elements to remain smooth during construction, making construction easier and faster.

7.3.1 Possible Usages

There are several situations in which mechanical splices are either required or more practical and cost-effective than lap splices. A few of the most prevalent conditions, as described in ACI 439.3, are:

a. **When Bar #14 and 18 (#43 and 57) are used.** ACI provisions allow lap splice only for bar #11 or smaller diameter bars. Therefore, when #14 and #18 bars are to be used, the only option is to use mechanical splices where needed. However, the provisions of ACI 18.2.7.2 must be satisfied.
b. **Reinforcing bars are too close to permit lap splicing.** This generally occurs in situations requiring large amounts of reinforcement and the use of larger bars, as in heavily loaded columns.
c. **Codes and specifications for tension lap splices result in long lap splice lengths.** This is the case, especially for bar sizes such as #8 and above in Grade 60 (420) or in a higher grade of steel or in epoxy-coated reinforcing bars.
d. **In tension tie members.** Mechanical splices are mandatory in tension tie members. Lap splices in such members are not permitted by code.

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e. **When the location of construction joints and provisions for future construction dictate the use of mechanical splices to provide tensile continuity.** Mechanical splices are often preferable to keeping long bar lengths projecting from existing concrete construction for future extension. A minimum of a 12 inches (305 mm) bar extension provides sufficient length for the application of most mechanical splices without damaging the existing concrete during installation. If mechanical splices must be staggered, the projecting bar extension should be greater to allow for the stagger pattern.
f. **To satisfy a steel-concrete ratio of 8% maximum for reinforced concrete columns.** This limit is considered in Chapter 10 of ACI 318 to be a practical maximum for reinforcement in terms of economy and requirements for placing. By using mechanical splices instead of lap splices, the amount of longitudinal steel may be significantly reduced in the splice region.
g. **When existing reinforced concrete structures are renovated, modified, or repaired and new reinforcing bars must be spliced to those already in concrete.** Mechanical splicing requires less removal of concrete compared to lap splicing.
h. **When members may be subjected to inelastic deformations due to earthquakes.** Strength capacities established in ACI 318 are higher for mechanical splices than for lap splices. Lap splices may not be reliable under conditions of cyclic loading into the inelastic range. Type 2 mechanical splices are qualified by tests to sustain inelastic reinforcing bar strain excursions provided that it conforms to ACI 318-19 Section 18.2.7.2.

All mechanical splices have particular physical features, in the splice components or in the required installation equipment or procedure, that can influence design and construction. Selection of splice type, components, and installation requirements, installed cost, and available options should be considered in the design process. The following subsections identify considerations for splice selection.

7.3.2 Some Popular Mechanical Splice Designs

A variety of mechanical arrangements, some of which are proprietary, are available to splice reinforcing bars. Physical characteristics of some of the popular types are described in the next paragraphs. No attempt has been made to include dimensional or other engineering data because information varies from each manufacturer and may change. ACI 439.3R-07 includes descriptions of a variety of mechanical splices.

7.3.2.1 Cold-Swaged Steel Coupling Sleeve

A cold-swaged steel coupling sleeve consists of a seamless steel sleeve that slips over the ends of two reinforcing bars and is forced deformed onto the reinforcing bar profile by means of a hydraulic jack to produce a mechanical interlock (Figure 7.5). Bar sizes #3 through 18 (#10 through 57) can be spliced by this method, including bars of different sizes. Coupling sleeves are available for use with rebar bar Grades

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40 (280), 60 (420), and 75 (520). When used with epoxy-coated bars that comply with ASTM A775/A775M, it is not necessary to shield the bar ends at the time of coating application or to remove the coating from the bar ends. Hot-dipped galvanized coupling sleeves can be ordered for galvanized bars that comply with ASTM A767/A767M. The cold-swaged splicing method is appropriate for new construction, field maintenance, and the splicing of older types of reinforcing bars, provided that such bars contain mechanical interlock deformations.

The first reinforcing bar is marked and inserted halfway into the sleeve. For field installation, a hydraulic press with a detachable two-piece die set is used. In a sequence of pressings that overlap one another, the die set exerts a radial force onto the reinforcing bar, which causes the first half of the coupling sleeve to be deformed. The second bar is inserted to abut against the first, and the remaining un-swaged portion of the sleeve is swaged onto the second bar. The total weight of a field-type press, including the dies, ranges from 20 to 230 pounds (9 and 105 kg). They can be supported for use in any orientation.

FIGURE 7.5 Cold-swaged steel coupling sleeve.

For field or shop use, larger bench-type presses with adjustable stops and insertion gauges are available. These machines can be used to swage a coupling sleeve onto the end of a reinforcing bar before placing it on the other bar. Adapter kits permit the use of the aforementioned field-presses in the same manner.

No particular bar-end preparation is required; thus, ends may be sheared, sawed, or flame-cut; nevertheless, a bar-end inspection is advised. Bars can be joined in any orientation, as the location of the press around the bar is not critical. In order to successfully install mechanical splices on bars with very close spacing, it is important to check with the manufacturer regarding the necessary equipment.

Cold swaging couplers are generally easy to install at the site for beam or slab rebar. However, for splicing column rebar, arrangements are required to erect and

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hold the hydraulic press at the desired location, which is often high up from the ground or floor level. The rebar to be joined also must be supported and kept in a vertical position by some means.

7.3.2.2 Cold-Swaged Coupler with Taper-Threaded Ends

This type of coupler consists of two pieces, both of which are manufactured from seamless steel tubing or carbon steel bar. The internally threaded coupler has a pre-cut taper thread inside one end; a preformed externally threaded coupler has a taper thread to match (Figure 7.6).

Straight bar ends can be sheared, sawed, or flame-cut; however, a bar-end check is suggested. Internal stops ensure that reinforcing bars are correctly positioned inside the couplers. A swaging technique using a bench press or a field-type press is used to distort the couplers into the bar profile to produce a mechanical interlock. Swaging can be done on the project site or at a fabricator's facility. When utilizing a bench press, most bar sizes require only one pressing per coupler half.

The tapered thread on the coupler helps to align the threads before rotating one of the two bars. The manufacturer requires that the threads be fully assembled. No special equipment or torque wrench is needed for this operation. A pipe wrench may be needed to overcome bar weight. Because threads are formed on the couplers and not on the reinforcing bar, the cross-sectional area of the bar is not reduced.

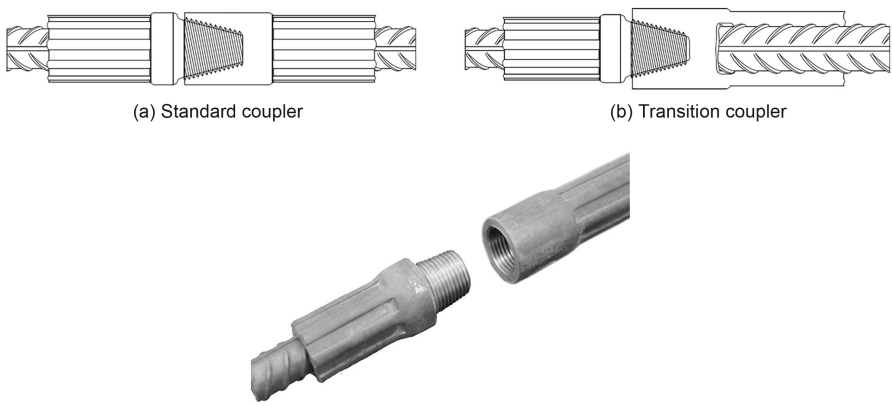

FIGURE 7.6 Cold-swaged steel coupler with taper threaded end (ACI 439.3R-07).

Bar Grades 40 (280), 60 (420), and 75 (520) in sizes #3 through 18 (#10 through 57), including transition sizes (Figure 7.6), can be spliced by either the two-piece or three-piece type couplers. When used with epoxy-coated bars that comply to ASTM A775/A775M, it is not necessary to shield the bar ends at the time of coating application or to remove the coating from the bar ends. Hot-dipped galvanized splice components can be ordered for mechanically splicing galvanized bars that comply with ASTM A767/A767M. An advantage of this method of coupling is that the cold swaging operation can be done on the ground before erecting the rebar to its proper location.

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7.3.2.3 Coupler for Thread-Like Deformed Reinforcing Bars

In this type of coupling system, the reinforcing bar needs to be manufactured with a special surface deformation pattern. The bars conform to ASTM A615 or A706 or other standard specifications, except for markings. These bars have rolled-on deformations having a profile similar to the shape of a helical thread, as shown in Figure 7.7.

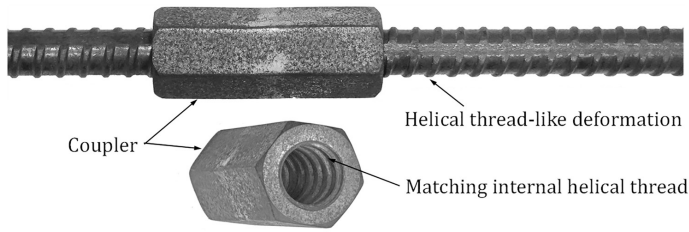

FIGURE 7.7 Thread bar reinforcement and coupler.

These reinforcing bars can be mechanically spliced using matching threaded couplers and nuts available from the reinforcing bar manufacturer. This mechanical splice is available for all grades up to Grade 75 (520) in bar sizes #6 through 18 (#19 through 57) and a special bar size #20 (#63), Grade 80 (550). The bars are mechanically spliced together using one of two methods,

1. A coupler is engaged on the ends of the two opposing bars in installations where one of the two bars can be rotated, and the two bars are tightened against each other. For mechanical splices working in compression, the bar ends should be perpendicular to within 1.5 degrees and may be either saw-cut or abrasion wheel-cut;
2. When neither of the connecting bars can be rotated, a coupler is engaged on the end of the opposing bars, and a jam nut is tightened against each end of the coupler, as shown in Figure 7.8. In this case, the ends will not have to be cut to the 1.5 degree perpendicularity tolerance.

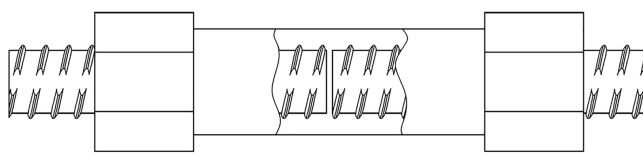

FIGURE 7.8 Coupler for thread-like deformed reinforcing bars (partial section) (ACI 439.3R-07).

For this type of mechanical splice, the assembly components are torqued together. Torque values range from 150 to 750 ft·lb (203 to 1016 N·m) for the #6 through 11

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(#19 through 36) sizes and are 1500 and 3000 ft·lb (2032 and 4065 N·m) for the #14 and 18 (#43 and 57) sizes, respectively. To attain higher torque values, hydraulic torque wrenches are available. Depending on the specific project requirements, required torque values may be less.

No special cleanliness requirements for the bar ends apply. Different size bars can be spliced. Bars should have a clear spacing of 1.5 bar diameters to provide room for tightening. As required, the manufacturer can supply compatible bars, threaded couplers, and nuts that have been epoxy coated or galvanized. It is not necessary to shield the bar ends at the time of coating application or to remove the coating from the bar ends.

7.3.2.4 Threaded Coupler with Upsized Bar Threads, Cold-Forged

This type of splice, for bar sizes #4 through 18 (#13 through 57), consists of two reinforcing bars with upsized parallel threaded ends that are connected by a coupler with internally machined threads (Figure 7.9).

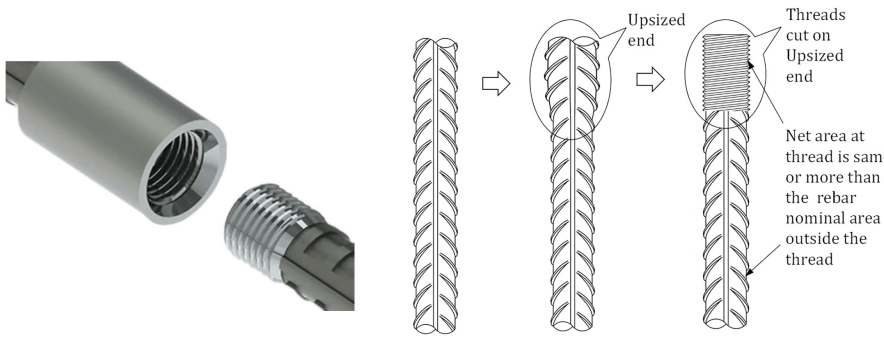

FIGURE 7.9 Threaded coupler with upsized bar threads, cold-forged.

Each reinforcing bar with a saw-cut end is inserted into a hydraulically actuated bench press that enlarges the bar end by a cold-forging operation. Thereafter, threads are formed on the upsized section of the bar by use of a thread-cutting machine or a thread-rolling machine. By virtue of the upsizing method, the nominal cross-sectional area of the bar in the threaded region is not reduced.

Bar end preparation can take place at a fabricator's plant or on the job site. A chamfer on the coupler helps locate the threaded ends for proper bar alignment. Assembly is completed by rotating either bar. This process does not necessitate the use of a torque wrench. The connection of reinforcing bars of different sizes is made by the use of a transition coupler with threads that match the corresponding threads on the reinforcing bars. Using standard threads on each bar makes it unnecessary to plan for which bar size will be used for a transition (Figure 7.10).

When neither reinforcing bar can be rotated, a position coupler is used by machining a longer thread on one of the bars to be connected. The threads extend from the upsized section of the bar into some of the reinforcing bar deformations. Assembly is completed by unwinding a preassembled coupler from the long thread onto the

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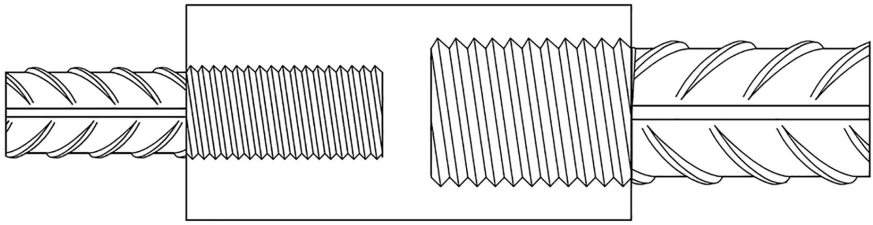

FIGURE 7.10 Transition coupler with upsized bar threads, cold-forged (ACI 439.3R-07).

adjoining threaded bar. An optional locknut is provided if one of the bars must be held in a specific orientation (Figure 7.11).

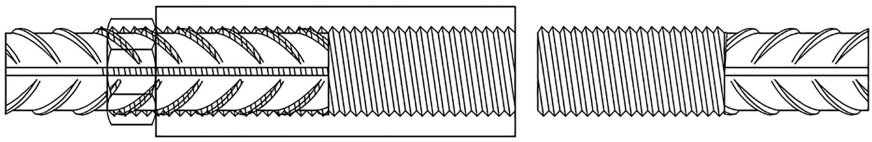

FIGURE 7.11 Position coupler with upsized bar threads, hot-forged (ACI 439.3R-07).

7.3.2.5 Shear Screw and Rail Coupling System

This type of device uses shear-head-type screws threaded into the side of seamless tubing and two serrated rails that are the same length as the coupling sleeve (Figure 7.12). The serrated rails cradle the reinforcing bar, and are embedded into the bar as the screws are tightened. The heads of the screws are designed to shear off at a prescribed torque to accomplish proper installation.

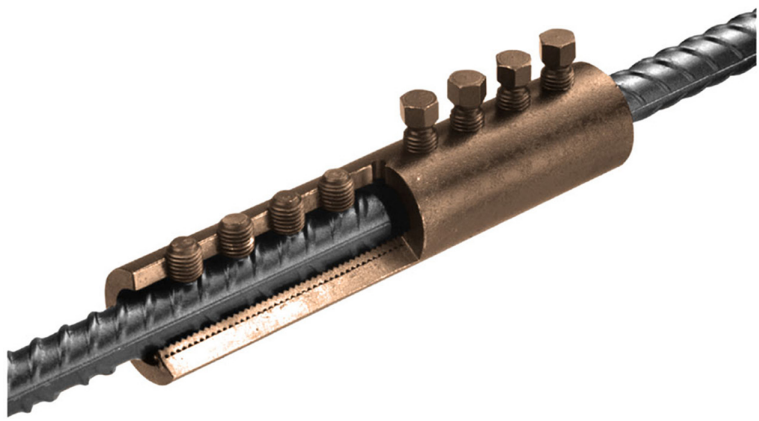

FIGURE 7.12 Shear screw and rail coupling system.

From each end, reinforcement bars are inserted. Ends can be sawed, flame-cut, or sheared and no special bar-end preparation is necessary. As the screws are tightened,

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they create an indentation in the surface of the bars, which then wedges into the serrated rail cradle. With the right impact wrench or hand-held ratchet wrench, screws can be tightened. At a specified tightening torque, the screw heads shear off indicating proper tightening.

7.3.3 Some Practical Considerations

Apart from meeting structural design requirements, there are a few issues related to fabrication and installation that must be taken into account while selecting and designing mechanical splices or couplers. These are discussed below.

7.3.3.1 Spacing and Cover Requirements

Minimum clear distances between adjacent reinforcing bars are specified in codes and design specifications. Chapter 25 of ACI 318-19 states that the clear distance limits also apply to adjacent bars in lapped splices, but do not address clearance limits for mechanical splices. The selection and positioning of the appropriate mechanical splice may be influenced by clearance limits for mechanical splices. The engineer/designer can determine whether the mechanical splices need to be staggered based on the clearance required by knowing the diameter of the mechanical splice. Mechanical splices are usually best located in the same plane or elevation for constructability reasons. Because, unlike lap splices, the ability of mechanical splices to carry and transfer load from bar to bar is not affected by concrete cover or concrete compressive strength, there is little justification for staggering mechanical splices solely to avoid undesirable failure modes associated with lap splices.

The size and operation of field equipment, if any, required for the installation of mechanical splices can sometimes dictate a minimum spacing or a staggered pattern. At splice locations, special attention should be paid to the minimum concrete cover of the stirrups, ties, or spirals. To avoid a reduced concrete cover, the stirrup or tie patterns adjacent to the mechanical splices, or the location of the splice itself, can be adjusted in many cases. Stirrups and ties are not required to be placed over mechanical splices according to codes. When a close spacing of stirrups and ties necessitates their placement over couplers or coupling sleeves, however, more cover over the splices and longitudinal reinforcement may be required to ensure stirrups or ties encompassing the mechanical splices have adequate cover. All structural components of mechanical splices must meet the minimum concrete cover dimension specified by the code. It's worth noting that flanges aren't structural in the case of dowel bar mechanical splices with flanges. As a result, the designer only needs to think about a cover over the flanges in terms of durability.

7.3.3.2 Matching and Preparation of End Detailing

Any special end preparation of bars required for a particular mechanical splice should be known to the designer. Chapter 25 of ACI 318, for example, specifies that the ends of the bars be cut to within 1.5 degrees of square with respect to the longitudinal axis for end-bearing splices.

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Some couplers are threaded by definition, and some require matching threads on the bar ends. Reinforcing bars with special thread-like deformations are required in one type of mechanical splice. In most cases, the mechanical splice should be concentrically aligned with the longitudinal bar axis for field assembly. This is especially important for couplers that require bar rotation reinforcement to ensure that the bar does not experience large oscillations during field assembly. By using bars with straight ends and securing the continuation bar in place at the time of installation, linear alignment from bar to bar can generally be preserved across the splice. The splice manufacturer's tolerances for angular misalignment and mis-insertion should be obtained. Coupling sleeve systems that are tolerant of misalignment should be used in situations where the axes of the reinforcing bars to be spliced are not concentrically aligned, such as curved bars, misplaced dowel bars, and precast-to-precast connections.

7.3.3.3 Equipment

The contractor and fabricator should work together to ensure that proper tooling and equipment are available on site to install selected mechanical splices on the specified reinforcing steel within the placing tolerance. A bar-end check is sometimes recommended before proceeding with certain mechanical splices. The manufacturer of the mechanical splice can provide advice on this.

7.3.3.4 Coated Reinforcing Bars

To prevent corrosion, reinforcing bars can be epoxy or zinc coated (galvanized). Similar techniques are used to make mechanical splices of coated and uncoated reinforcing bars. To properly install some types of mechanical splices on coated bars, the bar ends must be shielded at the time of coating, or the coating must be completely removed from the length of the splice device and a short distance beyond the ends of the splice device, approximately 2 inches (50 mm).

After mechanical splices are installed on coated reinforcing bars, the splice components and any damaged coating on the bars adjacent to the mechanical splices must be repaired with the appropriate compatible material. The project specifications should define the requirements for mechanical splice repair and coating restoration (refer to Chapter 5 of ACI 301). When possible, flame-cutting coated bars in any location should be avoided due to coating damage. Where mechanical splices are utilized, it may be more difficult to remove the flame-damaged epoxy coating.

7.3.3.5 Field Erection

Due to clearance, accessibility, and code requirements, mechanical splices can be staggered in many applications. If, for instance, staggered mechanical splices are used in columns, free-standing erection and assembly of the reinforcement may be required rather than preassembled cages, necessitating the use of external bracing or formwork to hold bars while completing splices.

There is a substantial difference between the equipment and the amount of time required to install various mechanical splices. Therefore, the field erection procedure

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and schedule must be coordinated with the mechanical splice selection and installation procedure. If specialized equipment is required, the supplier or splice manufacturer should be contacted for specific information regarding its size, weight, operation, and availability.

For certain projects, the engineer may deem it necessary to specify the mechanical splice types to be used. In addition, the method of construction may influence the types of mechanical splices that can be utilized most conveniently. Before construction begins, all involved parties must consider any special requirements of the chosen mechanical splice. Examining the following descriptions will help you determine these requirements. If necessary, the supplier or manufacturer should be able to provide information regarding special requirements.

7.3.3.6 Inspection and Quality Assurance

For all the splices described here, some type of installation inspection or quality assurance is needed to ensure satisfactory performance. Quality system program statements, certificates of compliance, or certified material test reports should be requested from mechanical splice suppliers. Mechanical splice manufacturers can provide installation acceptance criteria for use by engineers and field inspectors. Depending on the jurisdiction, a special inspection may be required in accordance with a building code or design specification.

Destructive tensile testing of sister splices (also known as companion splices), or a proportion of production pieces, can be specified by the engineer to verify that a particular splice will not prematurely slip or otherwise fail, particularly in critical locations where Type 2 splices are used. The frequency and manner of tensile testing may form part of the project specifications or may be prescribed in building codes. When mechanical splices are used in safety-related structures, such as nuclear containments, it may be mandatory to perform destructive tensile tests on as many as 1-in-50 splices. In some jurisdictions, two or three test specimens may be prepared and tested at the start of the splicing phase. Batch testing (or ongoing testing) is performed by some agencies on 1-in-500 splices or in accordance with instructions issued by the project engineer. ACI 318 makes no recommendations on the testing frequency of installed splices on a project.

7.3.4 ACI 439.3 Requirements and Specifications

In accordance with ACI 439.3, mechanical splices are primarily classified into two types as defined below.

Type 1: a mechanical splice that can develop at least 1.25 times the *specified yield strength* of the bar. Type 1 splices are used in elements where there is little concern for inelastic deformations and elevated tensile stresses from seismic events.

Type 2: a mechanical splice that conforms to Type 1 specifications and can develop the *specified tensile strength* of the spliced bars. Type 2 splices

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have proven, through accepted industry testing, the ability to develop the specified tensile strength of the spliced bars for resistance to elevated tensile stresses in the event of seismic loading.

The above definitions are better understood with the help of Figure 7.13 where a typical stress vs. strain diagram of a ASTM A615 Grade 420 rebar is shown by the solid curve. For this Grade 420 rebar, the minimum yield strength is 420 MPa. Thus, $1.25f_y$ is 525 MPa which is indicated by the dashed line *ab*. For a coupler to be passed as Type 1, the failure stress (maximum load divided by the nominal rebar area) of the coupled sample must be equal to or more than $1.25f_y$ or 525 MPa. In other words, the failure stress must lie above the line *ab* in Figure 7.13. The specified minimum tensile (ultimate) strength of Grade 420 bar is 620 MPa which is indicated by the line *cd* in Figure 7.13. For a coupler to be passed as Type 2, the failure stress of the coupled sample must be equal to or more than 620 MPa. In other words, the failure stress must lie above the line *cd* in Figure 7.13.

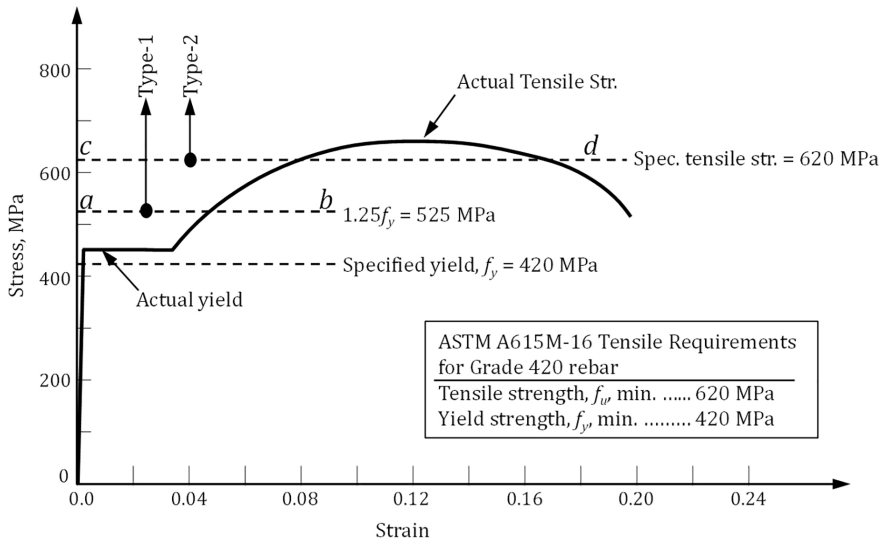

FIGURE 7.13 Definition of Type 1 and Type 2 mechanical splice in accordance with ACI 439.3 for ASTM A615M Grade 420 rebar.

Figure 7.14 illustrates the explanation of Type 1 and Type 2 couplers for ASTM A706M Grade 420 rebar. The explanation for a Type 1 coupler is the same as before. However, because of the different definitions of minimum tensile strength requirements in ASTM A706M, the explanation for Type 2 requirements is also different. Specification for A706 Grade 420 states that the minimum tensile strength shall be 550 MPa but it shall not be less than 1.25 times the *actual yield strength*. In

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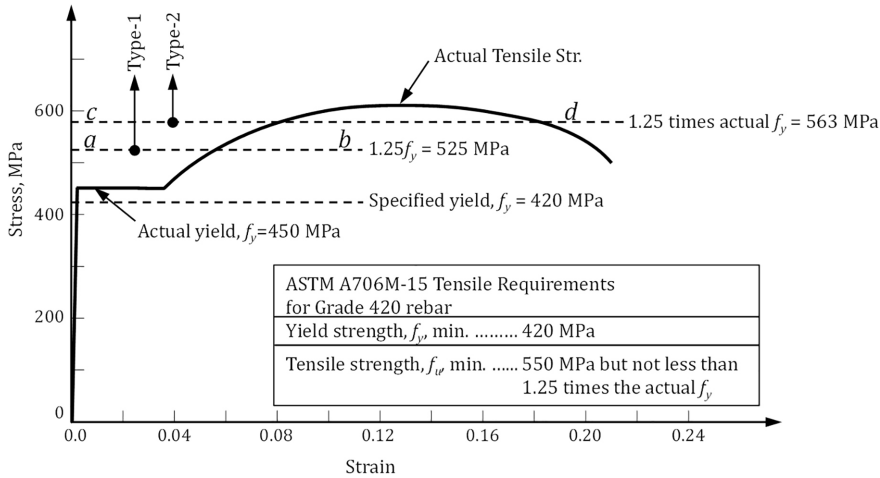

FIGURE 7.14 Definition of Type 1 and Type 2 mechanical splices in accordance with ACI 439.3 for ASTM A706M Grade 420 rebar.

Figure 7.14, the actual yield strength is 450 MPa. Thus the minimum tensile strength must be equal to or more than 1.25 × 450 = 563 MPa. Therefore, for a coupler to be qualified as Type 2, the failure stress of the coupled sample must be equal to or more than 563 MPa. In other words, the failure stress must lie above the line *cd* in Figure 7.14.

Design requirements of the applicable code (for example, ACI 318, 349, 350, or 359) or specification (for example, AASHTO Standard Specifications for Highway Bridges or AASHTO LRFD Bridge Design Specifications) for mechanical splices of reinforcing bars are not reproduced or discussed in detail in this book. Codes generally specify a minimum splice strength. For example, ACI 318-19, Chapter 25, requires that a (Type 1) full mechanical splice develops in tension or compression, as required, at least 125% of the specified yield strength, f_y, of the bar, so some yielding will occur in the reinforcing bar adjacent to the mechanical splice before failure in the splice itself. Code also requires that Type 2 mechanical splices develop the specified tensile strength of the spliced bar. For further background information and some of the considerations made in developing the ACI 318 mechanical splice provisions, the reader is encouraged to review the commentary provided in Chapter 25 of ACI 318-19.

Testing of mechanical splices in accordance with the American specification is described in detail in ASTM A1034/A1034M. The methods for testing, such as monotonic tension tests, monotonic compression tests, slip tests, and cyclic load tests are described in this specification but do not quantify the parameters for testing nor any acceptance criteria. Typical failure modes of couplers failing in tension are shown in Figure 7.15. In this Figure, the left most three samples failed by breaking within the couplers. In the right most three samples, the failure occurred by ultimate tensile fracture of the rebar. It is to be noted that the failure modes shown in Figure 7.15 should

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not influence the basis of acceptance or rejection of the mechanical splice system. According to ACI 439.3, acceptance or rejection of couplers shall be based on strength criteria as explained in the preceding paragraphs with the help of Figures 7.13 and 7.14.

FIGURE 7.15 Typical tensile failure modes of coupler.

7.3.5 ISO 15835 Requirements and Specifications

Requirements for mechanical splices are elaborately defined in the European ISO 15835 Part 2. The requirements are categorized as follows,

a. Requirements of strength and ductility under static force.
b. Slip requirements under static force.
c. High cycle fatigue requirements in the elastic range.
d. Low cycle fatigue requirements in the elasto-plastic range.

Of the four categories stated above, requirements (a) and (b) are defined as mandatory. That means these two basic requirements must be met by all the couplers. Mechanical splices meeting these two requirements are termed as Category B splices as defined in Annex-C of ISO 15835. Requirement (c) is relevant when the coupler is meant to be used in bridges or in similar applications where loading in large number of cycles at a low stress level is anticipated. Requirement (d) is relevant to the seismic application of couplers in buildings and structures.

7.3.5.1 Requirements of Strength and Ductility under Static Force

The strength and ductility requirements are the first mandatory requirements for mechanical splices in accordance with ISO 15835 standard. The strength requirement

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states that the coupler tensile or breaking strength shall be at least the specified minimum tensile-to-yield ratio times the minimum specified yield strength. The requirement can be expressed as:

$$\text{Tensile strength of coupler} \geq R_{\text{eH, spec.}} \times \left(\frac{R_{\text{m}}}{R_{\text{eH}}} \right)_{\text{spec.}} \tag{7.3}$$

The strength requirement is better explained with the help of Figure 7.16 where the requirements are illustrated for B420DWR rebar. For this rebar, the minimum specified yield strength, R_{eH}, is 420 MPa and the minimum $R_{\text{m}}/R_{\text{eH}}$ ratio specified is 1.25. Thus, according to Equation 7.3, the minimum breaking strength for mechanical splices shall be 420 × 1.25 = 525 MPa.

The ductility requirement states that the elongation at maximum force outside the length of the mechanical splice shall not be less than $0.7A_{\text{gt}}$, where A_{gt} is elongation at maximum force specified in ISO 6935-2. Where A_{gt} is not specified for the reinforcing bars, a minimum value of 3% should be reached in the bar outside the mechanical splice before failure of the test piece.

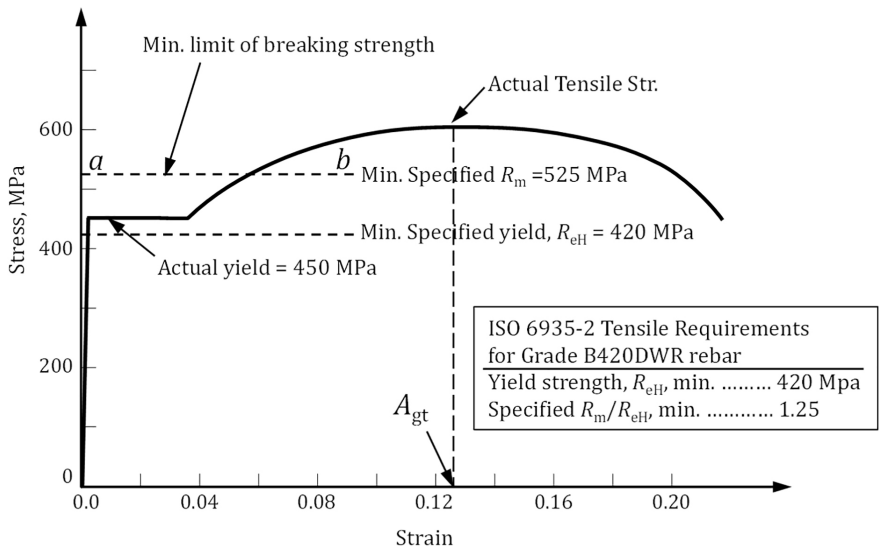

FIGURE 7.16 Strength requirement of mechanical splices in accordance with ISO 15835 for B420DWR rebar.

7.3.5.2 Slip Requirement under Static Force

Slip requirements under static force are the second mandatory requirements for all mechanical splices in accordance with ISO 15835 standard. The method of slip measurement is illustrated in Figure 7.17. The slip can be measured following Option 1 or Option 2 as described next.

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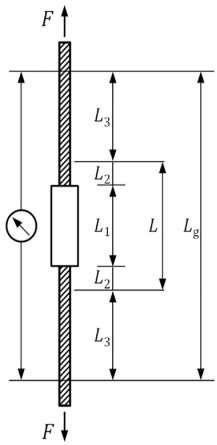

F Applied force

L Length of the mechanical splice as define in ISO 15835-1

L_1 Coupler length

L_2 $2d$ where d is the diameter of the rebar

L_3 in the range $2d$ to $3d$

L_g Overall gage length, in the range from $L_1 + 8d$ to $L_1 + 10d$

FIGURE 7.17 Slip measurement in a mechanical splice in accordance with ISO 15835.

Option 1: The slip across the mechanical splice shall be found as the measured length (gage length L_g in Figure 7.17) of the mechanical splice under a force corresponding to 60% of the minimum specified yield strength of the reinforcing bar minus the calculated length of an unspliced bar under similar force.

Option 2: The slip across the mechanical splice shall be found as the measured length (gage length L_g in Figure 7.17) of the mechanical splice after unloading from a load level of at least 60% of the minimum specified yield strength of the reinforcing bar minus the length prior to loading.

The requirement is that the slip found by either Option 1 or Option 2 shall not exceed 0.1 mm.

7.3.5.3 High Cycle Elastic Fatigue Requirements

ISO 15835 specifies high cycle fatigue requirements for mechanical splices though this is not termed "mandatory." Mechanical splices that need to meet the high cycle fatigue requirement are termed as of Category F as defined in Annex-C of ISO 15835. Such types shall sustain a fatigue loading of at least 2.0 million cycles with a stress range of 60 MPa without failure. The upper stress in the test shall be 60% of the minimum specified yield strength. A mechanical splice system can be considered satisfactory if all the samples pass the test without any crack or failure.

If two or more samples fail, the test is considered failed. If one sample fails, three additional similar samples need to be tested and pass.

7.3.5.4 Low Cycle Reverse Elastic-Plastic Fatigue Requirements

The objective of a low cycle reverse elastic-plastic fatigue test is to see the acceptability of the mechanical splice in seismic events. Two types of low cycle tests can be carried out – one simulating moderate-scale earthquakes and one simulating violent

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earthquakes. Splices for these types are termed as Category S1 and S2 respectively in accordance with Annex-C of ISO 15835. For further details regarding the test procedure and acceptance criteria, the reader is advised to see ISO 15835 Parts 1 and 2.

7.3.6 Design Considerations

Due to the minimum mechanical splice strength requirement, it is generally assumed in design that the presence of a mechanical splice of two reinforcing bars does not result in a reduction of the anticipated structural strength, the stiffness, or the ductility of the reinforcing steel. Design codes cover strength requirements for welded and mechanical splices, but generally do not specify how to avoid potential weaknesses that may be directly attributed to the specific details or materials of a mechanical splice. The following issues must be taken into consideration.

a. In a flexural member, the mechanical splice should not result in a low effective longitudinal stiffness of the reinforcement that violates the strain conditions assumed in the member design.
b. In sections where inelastic strains are anticipated, such as in potential plastic hinge sections (e.g. beam-column joints in special seismic resistant structures) the assembly of the mechanical splice and the reinforcing bars should possess adequate ductility so that premature failure of the spliced reinforcement is avoided.
c. A Type 2 splice may be used with Grade 60 rebar only in accordance with the provisions of ACI 318-19. The use of any mechanical splices with a rebar grade higher than Grade 60 is not allowed in seismic applications per ACI 18.2.7.2. Welded splices at these locations are not permitted by ACI 18.2.8.1.
d. ACI 439.3 is only concerned with strength characteristics. ASTM A1034 includes test procedures for tension, compression, slip, and fatigue characteristics. But neither of these provisions define any requirements regarding ductility, slip, and fatigue characteristics. Provisions of European ISO 15835 appear to be much more elaborate and comprehensive. The ISO 15835 comprehensively specifies strength, ductility, slip, and fatigue requirements for mechanical splices.

7.4 WELDED SPLICES

The use of welded splices for flexural reinforcement is generally permitted under reinforced concrete codes. The welding must comply with the provisions of AWS D1.4/D1.4M, which encompass issues of welding reinforcing bars and qualification standards for welding operations. Because ASTM A615, A616, and A617 do not provide restrictions on the chemical components that affect steel weldability, these specifications must be augmented to require a report of material qualities needed to conform to AWS D1.4/D1.4M requirements. ASTM A706 steel is designed for welding; hence such material property additives are unnecessary.

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A full welded splice, like mechanical splices, must be able to develop 125% of the specified yield strength of the bar. This is normally applicable for #6 and larger diameter bars. The exception given in Section 12.14.3.5 of ACI 318-19 is applicable in this case as well. AWS D1.4 also suggests that welded splicing of rebar should be avoided where the RC member would be subjected to impulse or impact loading.

7.4.1 Welding Process

Shielded metal arc welding (SMAW), gas tungsten arc welding (GTAW), gas metal arc welding (GMAW), and flux cored arc welding (FCAW) are the types of welding that are allowed when joining rebars. Other welding processes may be used when approved by a competent authority, provided that any special qualification test requirements not addressed by AWS D1.4 are met to ensure that welds are satisfactory for the intended application.

7.4.2 Design of Welded Splice

Design of welded splice follows the same basic principles of welded connection design. The loads and load combinations shall be as stipulated by the applicable building code. Either LRFD or ASD principle can be followed.

7.4.2.1 Weld Strength

The nominal strength, R_n, of the welded joint is determined according to the limit state of rupture as follows as given in Equation 7.4:

$$R_n = F_{nw} A_{we} \tag{7.4}$$

where

F_{nw} = nominal strength of weld metal, ksi [MPa] = 0.60 F_{EXX}
F_{EXX} = Tensile strength of weld material, ksi [MPa] = Usually 400 or 500 MPa
A_{we} = effective area of the weld, in^2 [mm^2].

Once the nominal strength is determined, the design strength in accordance with LRFD or ASD principles can be determined as follows.

LRFD	ASD
$\Phi R_n \geq R_u$	$R_n / \Omega \geq R_a$

where

R_u = required strength using factored load combinations
Φ = strength reduction factor = 0.75, Ω = safety factor = 2.0
ΦR_n = design strength of the joint in LRFD
R_n/Ω = allowable strength of the joint in ASD
R_a = required allowable strength based on ASD load combination.

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7.4.2.2 Effective Weld Areas, Lengths, Throats, and Sizes

For complete joint penetration (CJP) groove welds in direct butt joints, the effective weld area, A_{we}, shall be taken as the nominal cross-sectional area of the bar being welded (see Figures 7.18 and 7.19). The weld area shall be determined on the basis of smaller reinforcing bars when dissimilar reinforcing bar size numbers are being welded.

For flare-bevel and flare-v-groove welds, the effective weld area, A_{we}, shall be the effective weld length multiplied by the effective throat (see Figure 7.20). The effective weld length is the specified weld length. The minimum effective weld length shall not be less than two times the bar diameter for bars of equal diameter or two times the smaller bar diameter for bars of different diameters.

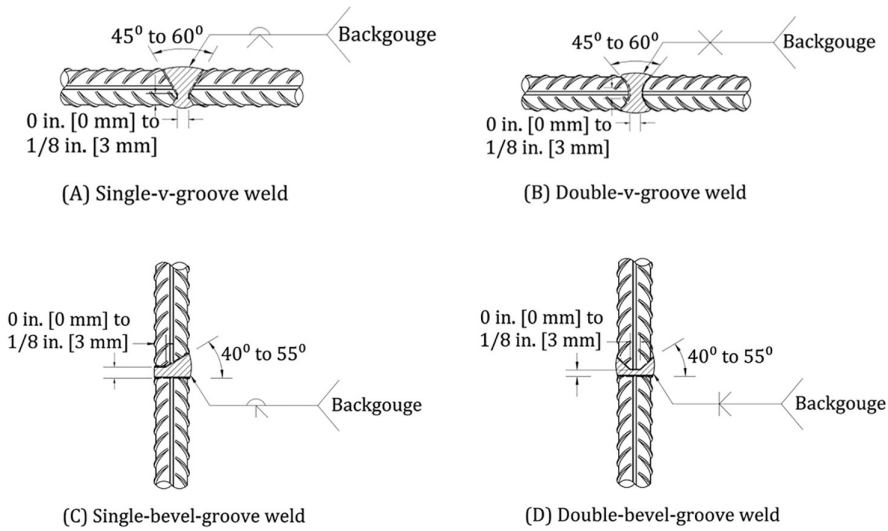

FIGURE 7.18 Direct butt joints (AWS D1.4/D1.4M 2018).

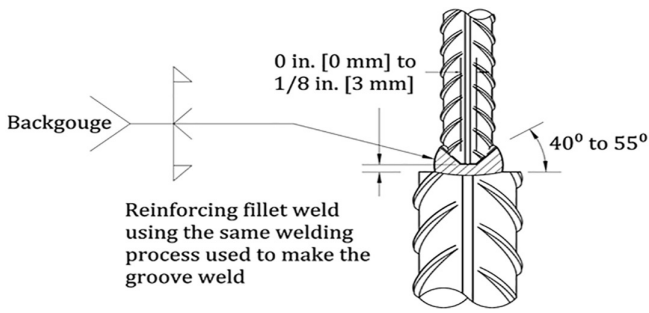

FIGURE 7.19 Direct butt joint showing transition between bars of different sizes (see 7.4.3) (AWS D1.4/D1.4M 2018).

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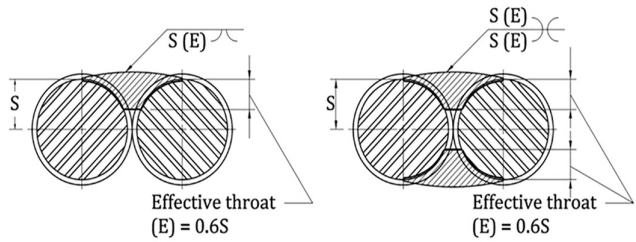
When filled flush with the solid section of the steel reinforcing bar, the effective throat shall be 0.4 times the bar radius for flare-bevel-groove welds and 0.6 times the bar radius for flare-V-groove welds. Larger effective throats may be utilized to determine the design strength if the WPS qualifies the larger effective throat. When joining bars of differing diameters, the effective throat must be based on the radius of the smaller bar.

For fillet welds, the effective weld area (A_{we}) is determined by multiplying the effective weld length by the effective throat. For any applied load direction, stresses in fillet welds shall be regarded as shear applied to the effective weld area. Along the weld axis, the effective weld length of a curved fillet weld must be measured. The dimension of the effective throat shall be computed as the minimum distance between the root of the joint and the face of the filled weld, minus the convexity.



Notes: [1]S = Nominal radius of steel reinforcing bar. [2]E = Effective throat. The throat is the least dimension from the point of fusion in the root the weld face, less any reinforcement.

FIGURE 7.20 Effective throat for flare-groove welds (see 7.4.2.2) (AWS D1.4/D1.4M 2018).

7.4.3 Structural Details

7.4.3.1 Butt Joints

Figure 7.19 shows how to make a direct butt joint between two different sized bars that are aligned along their lengths. Steel reinforcing bars can be welded with direct or indirect butt joints, lap joints, or T-joints (Figures 7.18, 7.19, 7.21, and 7.22). However, direct butt joints are better for bars larger than size #6 [19]. Figure 7.18 shows the type of joints that need to be welded. For bar sizes #8 [25] or smaller, one should adopt a single-bevel weld with back gouging and back welding.

Single or double flare-groove welds shall be used to create an indirect butt joint between the bars and the splice member (see Figure 7.21). The welding symbol without S and (E) dimensions indicates the weld shall be filled flush.

7.4.3.2 Lap Joints

Unless the joint can be accessed from only one side and has been approved by the engineer, a lap joint must be constructed using double-flare-V-groove welds (see Figure 7.22). Welds between the bars and the splice plate should be made using a single flare-bevel groove weld, with the bars separated (see Figure 7.22). Lap joints

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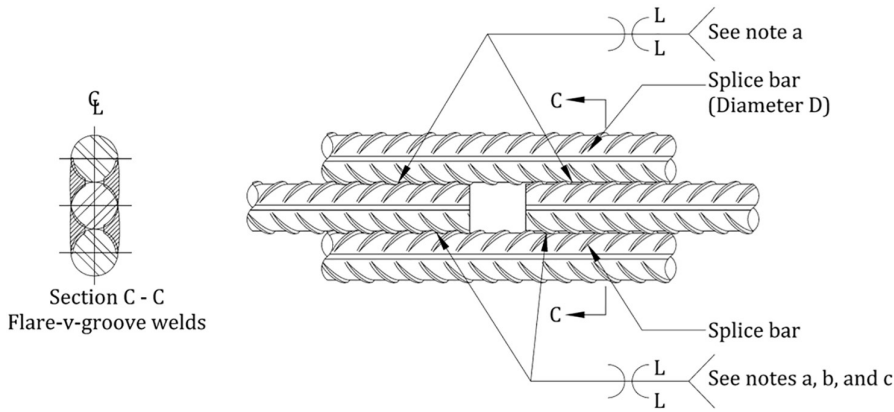
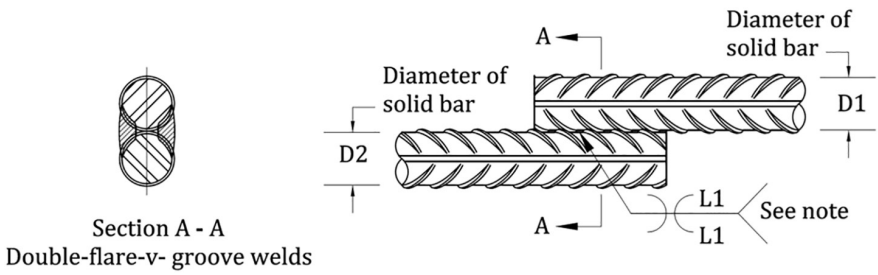


[a]L = 2D (min.). [b]Variation of this weld using single flare-V welds is permitted provided eccentricity is considered or restraint provided in the design of the joint. [c]Fill flush to bar tangent.

FIGURE 7.21 Indirect butt joints (AWS D1.4/D1.4M 2018).



Note: $L_1 = 2$ (D_1 or D_2) (min.).

FIGURE 7.22 Direct contact lap joint (AWS D1.4/D1.4M 2018).

welded with bar size #6 [19] or smaller are permitted. In the design of the joint, eccentricity effects must be taken into account, or appropriate restraint must be provided. Flush welds are indicated by the weld symbol without S and (E) dimensions.

7.4.4 Workmanship

7.4.4.1 Preparation of Base Metal

The base metal must be sufficiently clean to allow for the fabrication of welds that fulfill the quality standards of codes. Fins, tears, cracks, slag, or other base metal faults as described in the base metal standards are not permitted to be welded on. The surfaces to be welded, as well as the surfaces next to the weld, must be free of loose scale, thick scale, and thick rust. Welds may be done on mill scale and rust-covered surfaces if the mill scale and rust can be removed by vigorous hand wire

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brushing and the corresponding quality standards of the code are met. All surfaces at the place of weld and nearby locations shall be cleaned to remove excessive quantities of the following:

a. Water.
b. Oil or grease.
c. Other hydrocarbon-based materials.

Welding is allowed on surfaces with small amounts of foreign materials left on them, as long as the quality standards of welding (AWS D1.4) code can be met. As long as the quality requirements of the code can be met, it is okay to weld on surfaces that have surface protective coatings or anti-spatter compounds, except for those that are forbidden above. All coatings must be taken off within 2 inches (50 mm) of the faying surface of the joint.

Thermal cutting, air carbon arc cutting, sawing, or other mechanical means shall be used to shape the ends of steel reinforcing bars in direct butt joints to form the weld groove. The roughness of oxygen-cut surfaces shall not exceed the roughness limit of 2000 µin. [50 µm] prescribed in AWS C4.1 *Criteria for Describing Oxygen-Cut Surfaces*. Roughness greater than this value, as well as occasional notches or gouges no deeper than 3/16 in. [5 mm] deep on otherwise satisfactory surfaces, must be removed by machining or grinding. Bars for direct butt joints with sheared ends must be trimmed back beyond the sheared area.

7.4.4.2 Assembly

It is necessary to align the joint components in order to reduce eccentricities. Direct butt joints must not be offset at the joint by more than the following,

Bar sizes #10 [32] or smaller	⅛ in. [3 mm]
Bar sizes #11 [36] and #14 [43]	3⁄16 in. [5 mm]
Bar size #18 [57]	¼ in. [6 mm]

The maximum separation between the bar and the splice plate for indirect lap joints shall be no more than one-quarter of the bar diameter, but no more than $\frac{3}{16}$ inches [5 mm]. Cross-bar welding is not permitted unless authorized by the engineer.

7.4.4.3 Control of Distortion, Shrinkage, and Heat

In assembling and joining parts of a structure or precast member, the procedure and sequence followed shall minimize distortion and shrinkage.

When welding is done on bars or other structural parts that are already embedded in concrete, the steel's thermal expansion must be taken into account so that the concrete doesn't spall or crack or the bond between the concrete and the steel isn't destroyed. The heat of welding may cause localized damage to the concrete, which should be avoided.

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7.4.4.4 Quality of Welds

Welds that do not meet the quality requirements as discussed in the following paragraphs shall be repaired by removal of unacceptable portions or by rewelding, whichever is applicable.

The fillet weld faces shall be slightly convex or slightly concave as shown in Figure 7.23, or flat, and with none of the unacceptable profiles exhibited in Figure 7.23. Any crack, no matter how big or where it is, is not acceptable. There must be a full fusion between the weld metal and the base metal, as well as between each pass in the weld. All weld craters must be filled until the cross section meets the minimum weld size specified. There shall be no overlap in the welds. In the solid section of the bar or structural member, undercuts with a depth greater than 1/32 inches [1 mm] are prohibited.

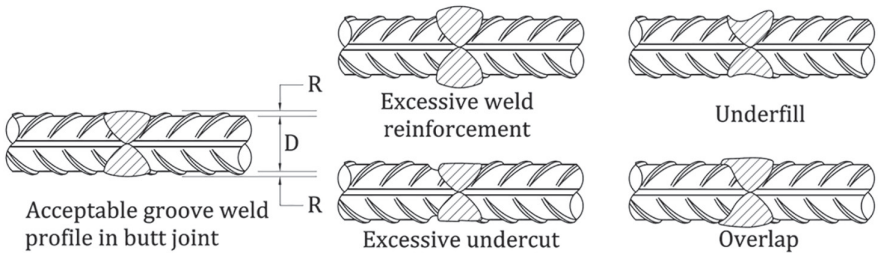

FIGURE 7.23 Acceptable and unacceptable weld profiles (AWS D1.4/D1.4M 2018).

TABLE 7.2
Radiographic acceptance criteria[a] (see 7.4.4.4) (AWS D1.4/D1.4M 2018).

Bar Size Number	Sum of the Maximum Dimensions of all Porosity or Fusion-Like Discontinuities		Maximum Dimension of Any Single Porosity or Fusion-Type Discontinuity	
	in.	mm	in.	mm
8 [25]	3/16	5	1/8	3
9 [29]	3/16	5	1/8	3
10 [32]	1/4	6	1/8	3
11 [36]	1/4	6	3/16	5
14 [43]	5/16	8	3/16	5
18 [57]	7/16	11	1/4	6

[a]Acceptance criteria for bar size numbers less than size #8 [25] shall be established by the engineer.

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The size of the weld must be the same as or bigger than the size of the weld specified. (Effective throat (E) dimension is specified, see Figure 7.20 for required size.) The length of the weld with this weld size must be the same as or longer than the length of the weld specified. Any portion of the length, including starts or stops, that contains a smaller weld size shall not be measured in the weld length.

In the case of radiographic inspection of direct butt joints, the maximum size of any one porosity or fusion-like discontinuity, or the sum of the maximum dimensions of all such discontinuities, shall not exceed the limits specified in Table 7.2.

Repairs to welds made with the shielded metal arc welding (SMAW) and gas metal arc welding (GMAW) processes shall be made in accordance with a qualified WPS approved by the engineer.

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QUESTIONS

7-1. Why is rebar splicing essential? Describe the most common methods of rebar splicing.

7-2. Under what conditions should Class A lap splices be permitted? Where are non-contact lap splices allowed, and how can they be applied?

7-3. What is mechanical splicing? Briefly discuss its various features.

7-4. How does a mechanical splice reduce material usages?

7-5. Write the possible usages of mechanical splices.

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7-6. Describe the working principle and installation procedure for cold-swaged steel coupling sleeves.
7-7. Provide a brief overview of couplers designed for thread-like deformed reinforcing bars.
7-8. Write the fabrication procedure for threaded couplers with upsized bar threads.
7-9. Write the working principle of a shear screw and rail coupling system.
7-10. What are the requirements for spacing and cover in mechanical splicing?
7-11. Explain the process for preparing the end detailing of couplers.
7-12. Briefly discuss the installation procedure for couplers on coated reinforcing bars.
7-13. Describe and explain the classification of mechanical splices according to ACI 439.3, and illustrate your explanation with figures.
7-14. Explain the testing procedure for mechanical splices as specified by ASTM standards.
7-15. Briefly describe the categories of the requirements of ISO 15835 specifications for mechanical splices.
7-16. Explain, with neat sketches, the strength and ductility requirements of mechanical splices in accordance with ISO 15835 standard.
7-17. Explain, with neat sketches, the slip requirement of mechanical splices in accordance with ISO 15835 standard.
7-18. Discuss the high cycle and low cycle fatigue requirements for rebar in accordance ISO 15835 standard.
7-19. What are the design considerations for using mechanical splices in concrete structures? Discuss.
7-20. Briefly discuss, including the exceptions, the applicability of welded splices for rebar in concrete structures.
7-21. With neat sketches, describe the different types of weld geometry that may be adopted in a direct butt joint.
7-22. Discuss the different aspects of workmanship that must be maintained to obtain a good quality weld splice.
7-23. With neat sketches, describe acceptable and unacceptable weld profiles.

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8 Stainless Steel Rebar

8.1 INTRODUCTION

Reinforcement in concrete structures is generally protected chemically and physically by concrete. Aside from issues caused by poor design and construction, carbonation of concrete causes the passivation of the oxide layer to become unstable (i.e. de-passivation). Carbonation of concrete reduces the alkalinity of the pore solution in the hardened cement paste around the steel and allows chlorides to infiltrate the steel-concrete interface (Nürnberger 1995; Cigna *et al*. 2003). When long life is required, or corrosion is expected, the designer has several traditional options. Stainless steel based reinforcement, as shown in Figure 8.1 is one appealing technical solution (Nürnberger 1996; Bauer and Cochrane 1999; Hunkeler 2000; Nürnberger 2005).

Stainless steel is a broad category of steel types with a wide range of physical and mechanical properties, as well as cost and corrosion resistance. They have

FIGURE 8.1 A bundle of stainless steel rebar. [©FHWA, background removed by the author, Wikimedia Commons CC0 1.0, original available at https://commons.wikimedia.org/wiki/File:Solid_Stainless_Steel_Rebars_(8426557131).jpg, accessed 25 September, 2024].

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significantly better corrosion resistance than carbon steel, thanks to a chromium-rich passive film on their surface. Structures exposed to aggressive environments, particularly in the presence of chlorides, can benefit from stainless steel bars as a preventative measure. They can also be used to repair corroding structures and in specific parts of structures where corrosion is most likely to occur. Engineers can choose from a variety of stainless steel types to find the best combination of strength, corrosion resistance, and cost. Some of the advantages of using stainless steel rebar are as follows (Stainless Rebar 2022):

a. It is extremely resistant to chloride ion corrosion.
b. It does not rely on concrete's high alkalinity for protection.
c. The amount of cover to concrete can be reduced.
d. Use of concrete sealants like Silane can be avoided.
e. Concrete mix can be simplified to meet the needs of concrete design rather than rebar protection.
f. It increases longevity.
g. Maintenance and repair costs are reduced.
h. It can be used selectively and cost-effectively for high-risk elements.
i. It will be recycled at some point.

8.2 STAINLESS STEEL REBAR VS. OTHER TYPES

Several alternative solutions such as a) epoxy coating of rebar, b) galvanizing of rebar, c) non-metallic rebars such as FRP or basalt fiber rebar, and so on have emerged to address the corrosion problems of mild steel (carbon steel) rebar used in concrete, as well as associated concrete durability issues. Steel corrosion is sometimes reduced by using cathodic protection. In some constructions, a waterproof membrane is used to prevent water from getting in. Epoxy coating, galvanized rebar, and non-metallic rebar are discussed in later chapters. Some details on the manufacturing process, properties, and applications of these various types of coated or non-metallic rebar are discussed there. Table 8.1 provides a quick comparison of some of these different approaches and methodologies.

8.3 MATERIALS

Stainless steels are high alloys containing chromium (Cr) and nickel (Ni). Standardized stainless steels have a minimum chromium content of 10.5%. Low-chromium steel should not be referred to as "stainless." Chromium is the primary alloy that improves the corrosion resistance of steel (Markeset *et al.* 2006). This enhanced corrosion resistance is depicted in Figure 8.2. The enhanced corrosion resistance is attributable to a thin chromium oxide film that forms on the surface of the steel and creates a so-called passive condition. It is essential to realize that oxygen is required for the formation of the oxide film. Passivity is a dynamic process affected by the surrounding environment, particularly temperature and humidity. In

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TABLE 8.1
Comparison of different approaches to prevent rebar corrosion

Method	Advantages	Drawbacks
Epoxy coating	Lower initial costs	a. Difficult to bend without cracking b. Requires careful handling to avoid damage during transportation, storage and installation c. Careful storage away from sunlight needed to avoid disintegration of the epoxy due to UV radiation d. Causes bond or lap length to increase.
Galvanizing	Lower initial costs	a. Zn coating is sacrifical and is no longer effective once zinc has corroded
Fiber-reinforced polymers	Lower initial costs Lighter	a. Cannot be bent at site b. In harsh winters, there is no heat resistance and little impact resistance c. Properties changes over time (aging) d. Degradation due to temperature variation e. Disposal at end of life is expensive due the hazardous nature of the fiber particle
Cathodic protection	Lower initial costs	a. Overall protection necessitates careful design b. To maintain proper electrical contacts, careful installation is required c. Requires a constant current source (which must be monitored and maintained) or sacrificial anodes that must be monitored and replaced d. Requires continuous monitoring which may be expensive
Membranes	Lower initial costs	a. Requires careful installation b. Performance over time is debatable c. Limited to horizontal surfaces
Stainless steel	a. Life cycle cost is low b. Stainless steels have a similar design concept as that of carbon steel c. Mixed carbon steel/stainless reinforcements are effective d. Tolerates poor craftsmanship (insufficient concrete cover, poor compaction, inadequate curing) e. Not very sensitive to poor concrete quality f. Installation is simple g. There are virtually no maintenance costs h. There is no life limit i. Concrete cover can be thinner j. Improved fire resistance k. It is completely recyclable l. Removes corrosion and, as a result, corrosion-induced concrete cracking	Initially more expensive, but only a few percent of the total cost of the structure

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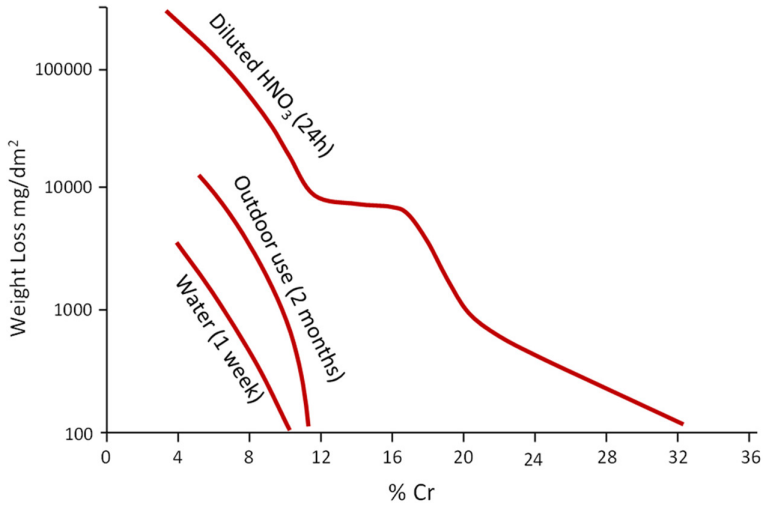

FIGURE 8.2 Influence of chromium on the corrosion resistance of stainless steel (Markeset *et al.* 2006).

the presence of oxygen, the extremely thin chromium oxide film is also capable of self-repair.

In addition to chromium, common alloying components include molybdenum, nickel, and nitrogen. Stainless steel is typically alloyed with nickel to improve its formability and ductility. Different crystal structures are produced by alloying these elements, allowing the steel to have different properties for machining, forming, welding, etc. The four principal types of stainless steel are as follows (Markeset 2008):

a. Austenitic.
b. Ferritic.
c. Austenitic-ferritic (duplex).
d. Martensitic and precipitation hardening stainless steel.

The most common type of stainless steel is austenitic stainless steel. It contains at least 7% nickel, making the steel structure fully austenitic and providing ductility, a broad temperature range, non-magnetic properties, and good weldability. Austenitic stainless steel offers a wide range of applications, including housewares, containers, industrial piping and vessels, architectural facades, and building structures. For reinforcement, such steels are currently rated in the higher range of corrosion resistance.

Properties of ferritic stainless steel are similar to that of mild steel, but are more corrosion resistant. The most common chromium-containing steels are those with 12% and 17% chromium, with the former used primarily in structural applications and the latter in housewares, boilers, washing machines, and indoor architecture.

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For reinforcement, such steels are currently rated in the lower range of corrosion resistance (Markeset 2008).

Austenitic-ferritic (duplex) stainless steel has a combined ferritic and austenitic lattice, hence its common name: duplex stainless steel. This steel's lattice structure is partially austenitic due to its nickel content. The duplex structure provides both durability and pliability. The petrochemical, paper, pulp, and shipbuilding industries are the most common users of duplex steels. Due to fluctuating prices of key alloying elements, duplex steels may occasionally be more economical than austenitic steels. In such instances, duplex steels become desirable as concrete reinforcement in the construction industry. In environments with high chloride concentrations and high temperatures, duplex steel reinforcement is also advantageous. These steels are currently rated in the extremely high range for corrosion resistance.

In applications requiring a precise, hardened edge, martensitic and precipitation hardening steels are popular, despite being the least common of the four major families of stainless steel. Due to its ability to temper and harden, this family is a top choice for knives, scissors, razors, and medical instruments. The corrosion resistance of these stainless steel alloys ranges from moderate to good, and they retain their magnetic properties after hardening. This type of steel is not considered for rebar production (Markeset 2008).

A comparison of different characteristics of stainless steel alloys is given in Table 8.2. Growth in the use of stainless steel reinforcement to solve the corrosion problem at its source has resulted from the growing recognition of the negative societal impact of needing to repair prematurely corroded concrete structures, as well as the growing environmental impact of such repairs and the growing recognition of the negative effects on user convenience caused by repair works. This awareness, coupled with the high costs of traditional stainless steel – which were originally developed to solve corrosion problems in areas other than reinforced concrete

TABLE 8.2
General comparison of different alloys of stainless steel

	Alloy group				
	Austenitic	**Duplex**	**Ferritic**	**Martensitic**	**Precipitation hardening**
Magnetic response	Generally no	Yes	Yes	Yes	Yes
Work hardening rate	Very high	Medium	Medium	Medium	Medium
Corrosion resistance	High	Very high	Medium	Medium	Medium
Hardenable	By cold work	No	No	Quench and temper	Age hardening
Ductility	Very high	Medium	Medium	Low	Medium
High temperature resistance	Very high	Low	High	Low	Low
Low temperature resistance	Very high	Medium	Low	Low	Low
Weldability	Very high	High	Low	Low	High

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structures – has prompted stainless steel manufacturers to search for methods of producing durable stainless steel that is less susceptible to the price volatility of key alloying components in recent years. This has resulted in the development of low-nickel duplex types (lean duplex) with a sufficient nickel content to maintain the highly corrosion-resistant austenitic-ferritic crystal structure, but with a significantly reduced nickel content. Although none of these products are in large-scale production, their corrosion resistance and mechanical properties appear to be quite promising (Markeset *et al.* 2006).

8.4 CLASSIFICATION AND CHEMICAL COMPOSITION

8.4.1 Traditional Classification System

Traditionally, stainless steels have been classified in different styles in different places. The American Iron and Steel Institute (AISI) categorizes ferritic and martensitic steels as 400 series alloys, such as 403 for ferritic steel. The austenitic steels are classified as alloys of the 300 series, such as 304 and 316. The types of these steel grades did not provide any additional information regarding the chemical composition or physical and mechanical properties. Historically, British standards, such as BS 6744, etc., have followed the AISI classification with the addition of "S" subgrades, such as Grade 316S33. However, British standards are being replaced by European ones, and those pertaining to stainless steel will adopt the current European classification for steels, as described below. The German or DIN classification is based on the idea of a material number, for instance, 1.44xx. X18Cr8Ni3Mo is an austenitic stainless steel with a nominal alloy composition of 18% chromium, 8% nickel, and 3% molybdenum. This is according to the French classification based on a unique material number for a given steel. Despite being a somewhat cumbersome designation, this classification provides nominal compositions for each type of steel (Markeset 2008).

8.4.2 European Classification System

The year 1995 saw the publication of a new European standard EN 10088-1 that standardized the classification of stainless steels. The standard effectively adopted both the German and French systems. Thus, each stainless steel now has a generic number that identifies its grouping and an individual material number referred to as its name that specifies the nominal composition of the alloy. The designation system can be understood following an example of a stainless steel classified as (Markeset 2008):

Material number: 1.4436

Material name: X3CrNiMo 17-13-3

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The material number has the following components:

- The first 1 Denotes a steel.
- The 44 after the dot denotes a stainless steel group.
- The next 36 indicates individual material characteristics.

The material name constitutes components as follows:

- A high-alloy steel is denoted by X.
- Digit 3 represents the carbon content in one-hundredths (in this case 0.03%).
- Chemical symbols of the main alloying elements are indicated – CrNiMo.
- The percentage of the main alloying elements are represented by a number in respective order, such as 17-13-3.

In order to compare stainless steel grades with various alloys, the influence of different elements has been correlated, resulting in the expression of a pitting resistance equivalent number (PREN). This expression can be used to rank corrosion resistance against chloride pitting corrosion as a relative measure of the steel grade's total resistance. The expression is derived from the alloying element content in the steel grade. The expression for austenitic steels is:

$$\text{PREN} = \%\text{Cr} + 3.3 \times \%\text{Mo} + 16 \times \%\text{N}$$

The effect of nitrogen is considered higher for duplex steels resulting in the expression:

$$\text{PREN} = \%\text{Cr} + 3.3 \times \%\text{Mo} + 30 \times \%\text{N}$$

A lower PREN value indicates higher susceptibility to pitting corrosion. This system appears to be more complicated than the AISI system it is supposed to replace. It is, however, more logical and provides insight into the alloy composition and, as a result, the material type within the classification.

Although there are many different types of stainless steels, only a few are considered to be generally useful as concrete reinforcement. For this reason, only the most common types of concrete rebar are discussed here. Table 8.3 compares the old and new classification methods for common stainless steel grades, as well as the pitting resistant equivalent number. It is obvious that increasing the amount of alloying elements, particularly chromium, nickel, and molybdenum, improves corrosion resistance. Changing the balance of alloying elements, on the other hand, will affect the structure as well as other properties. As a result, members of the stainless steel family are typically grouped together based on their metallographic structure. Table 8.4 shows the chemical compositions of stainless steel grades. The materials are arranged in tables in order of increasing corrosion resistance, as indicated by the PREN values. Corrosion resistance is improved by increasing alloy content, particularly Cr, Ni, and Mo, while material costs rise. The cost of these critical alloying elements varies depending on market conditions (Markeset 2008).

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TABLE 8.3
Classification of stainless steel according to international standards and corresponding PREN values (Markeset *et al.* 2006)

Steel type	Steel grade		USA	Great Britain	Sweden	PREN-value
	EN 10088-1	Designation	AISI	BS	SS	
Austenitic	1.4301	X5CrNi 18-10	304	314S11 /314S15	2332	19
Austenitic	1.4401	X5CrNiMo 17-12-2	316	316S33	2347	25
	1.4429	X2CrNiMoN 17-13-3	316LN	316S63	2375	26
	1.4436	X5CrNiMo 17-12-2	316	–	2343	26
	1.4571	X6CrNiMoTi 17-12-2	316Ti	–	2350	25
Ferritic-austenitic (lean duplex types)	1.41xx (LDX 2101)	X3CrNiMo 22-2-0	–	–	–	26
	1.4362	X2CrNiMo 23-4	–	–	–	24
Ferritic-austenitic (duplex)	1.4462	X2CrNiMoN 22-5-3	–	318	2377	36

8.4.3 Unified Numbering System (UNS)

The Unified Numbering System (UNS) is a North American composition-based system for commercial materials. A letter precedes each metal, which is followed by five numbers. Each UNS number refers to a specific metal or alloy and specifies its chemical composition, as well as mechanical or physical properties in some cases. The UNS system does not guarantee precise composition, impurity limits, or performance specifications. The letter "S" precedes five numerical digits in the designation of heat and corrosion-resistant (stainless) steels. The first three digits frequently correspond to older three-digit numbering systems, while the last two digits denote more modern variations. For example, in the original three-digit system, Stainless Steel Type 310 became S31000 in the UNS System. In the UNS System, the more modern low-carbon variation, Type 310S, became S31008. A material property specification is frequently represented by the suffix digit. Because the maximum allowed carbon content is 0.08%, "08" was assigned to UNS S31008.

The ASTM 955 standard covers deformed and plain stainless steel bars for concrete reinforcement in applications that require corrosion resistance or controlled magnetic permeability in the United States. The chemical requirements of typical alloys are shown in Table 8.5.

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TABLE 8.4

Chemical composition of stainless steel of relevance as reinforcement (Markeset *et al.* 2006)

	Steel grade		Chemical composition									
Type	EN 10088-1	Designation	C max	Si max	Mn max	P max	S max	Cr min/ max	Ni min/ max	Mo min/ max	N min/ max	Ti min/ max
Austenitic	1.4301	X5CrNi 18-10	0.07	1.0	2.0	0.045	0.03	17.5/ 19.5	8.0/ 10.50	–	Max 0.11	–
Austenitic	1.4401	X5CrNiMo 17-12-2	0.07	1.0	2.0	0.045	0.03	16.5/ 18.5	10.0/ 13.0	2.00/ 2.50	Max 0.11	–
	1.4429	X2CrNiMoN 17-13-3	0.03	1.0	2.0	0.045	0.015	16.5/ 18.5	11.0/ 14.0	2.5/ 3.0	0.12/ 0.22	–
	1.4436	X5CrNiMo 17-12-2	0.05	1.0	2.0	0.045	0.03	16.5/ 18.5	10.5/ 13.0	2.5/ 3.0	Max 0.11	–
	1.4571	X6CrNiMoTi 17-12-2	0.08	1.0	2.0	0.045	0.03	16.5/ 18.5	10.5/ 13.5	2.0/ 2.5	–	5xC/ 0.7
Ferritic-austenitic	1.41xx (=LDX 2101)	X3CrNiMo 22-2-0	0.03	0.4	5.0	–	–	21.5	1.5	0.3	Max 0.22	–
	1.4362	X2CrNiMo 23-4	0.03	1.0	2.0	0.035	0.015	22.0/ 24.0	3.5/ 5.5	0.1/ 0.6	0.05/ 0.20	–
Ferritic-austenitic (duplex)	1.4462	X2CrNiMoN 22-5-3	0.03	1.0	2.0	0.035	0.015	21.0/ 23.0	4.5/ 6.5	2.5/ 3.5	0.10/ 0.22	–

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TABLE 8.5
Chemical requirements of typical alloys in ASTM A955

		Composition%								
UNS designation	**Type**	**C**	**Mn**	**P**	**S**	**Si**	**Cr**	**Ni**	**Mo**	**N**
S24000	XM-29	0.08	11.50–14.50	0.060	0.030	1.00	17.00–19.00	2.25–3.75	…	0.20–0.40
S24100	XM-28	0.15	11.00–14.00	0.060	0.030	1.00	16.50–19.00	0.50–2.50	…	0.20–0.45
S30400	304	0.08	2.00	0.045	0.030	1.00	18.00–20.00	8.00–10.50	…	0.10
S31603	316L	0.03	2.00	0.045	0.030	1.00	16.00–18.00	10.00–14.00	2.00–3.00	0.10
S31653	316LN	0.08	2.00	0.045	0.030	1.00	16.00–18.00	10.00–14.00	2.00–3.00	0.10–0.16
S31803	…	0.03	2.00	0.045	0.020	1.00	21.00–23.00	4.50–6.50	2.50–3.50	0.08–0.20

8.5 PHYSICAL PROPERTIES

The density of stainless steel rebar (SSR) varies only marginally from normal carbon steel reinforcement and, as seen in Table 8.6, small variations may not be a concern in any practical application. In comparison to carbon steel, which has a thermal expansion of 12×10^{-6}/°C, SSR has a thermal expansion of approximately 16×10^{-6}/°C for austenitic steels and approximately 13×10^{-6}/°C for austenitic-ferritic duplex steels. Although there is a significant difference in thermal expansion between austenitic and carbon steel, this is not a factor in structural design. The issue is comparing the thermal coefficient of steel and concrete; traditional concretes have a coefficient that is similar to that of carbon steel. However, depending on the type and quality of aggregates (with granite and gabbro on the low end and limestone on the high end in terms of thermal expansion), the thermal expansion coefficient of the concrete can vary by as much as 20%, and the elastic modulus of the concrete can vary by as much as 20% or more, depending on the mix. Variations in strain properties in concrete structures have never been reported to cause structural or performance issues (Markeset 2008).

TABLE 8.6
Physical properties of stainless steel

Steel grade	Density [kg/m³]	Thermal expansion coefficient [10⁻⁶/°C]
Carbon steel	8000	12
1.4301	7900	16
1.4436	8000	16
1.4462	7800	13

Austenitic stainless steels are considered practically non-magnetic in general. Cold drawn bars, on the other hand, gain some magnetic permeability. Magnetic permeability falls in the following designation order: 1.4301 > 1.4436 > 1.4429 > 1.4529. As a result, SSR with low magnetic permeability (non-magnetic) must be hot-rolled and of a specific composition or alternatively tested. Magnetic permeability of typical austenitic stainless steel is of the order of $\mu = 1.260 \times 10^{-6}$ to 8.8×10^{-6} H/m. On the other hand, carbon steels possess magnetic permeability value of the order of $\mu = 1.26 \times 10^{-4}$ H/m. Carbon steels and duplex stainless steels are both magnetic (Markeset *et al.* 2006).

8.6 MECHANICAL PROPERTIES OF STAINLESS STEEL REINFORCEMENT

8.6.1 Stress-Strain Relationships

Figure 8.3 depicts the stress-strain relationship for various stainless steel types (Markeset 2008). Only austenitic and ferritic-austenitic (duplex) steels are appropriate

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for reinforcement. Steels in the austenitic and duplex (and ferritic) grades show early plastic deformation in tests and continue to withstand increasing loads as strain increases. Cold working increases the steel's strength and is thus used to meet the requirements for use as concrete reinforcement. Cold working usually results in martensite formation in 1.4301 materials, but not in 1.4401/1.4436 or duplex materials. Cold working reduces elongation from 40% to 20% to 25% for austenitic steels.

Warm working (at a temperature slightly lower than normal for such a process) can be used to increase strength for small dimensions (less than 16 mm), resulting in mechanical properties similar to those obtained by cold working.

Nitrogen addition (0.15–0.2%) is another way to boost strength. However, this is insufficient to achieve the required strength, so it must be combined with either cold or warm working. Proof strength is defined and determined as the tensile stress at 0.2% elongation to characterize the design strength of such strain hardening materials (strain 0.002). The maximum load that the tested reinforcement can withstand is defined as ultimate tensile strength. Within the standard range of strength and

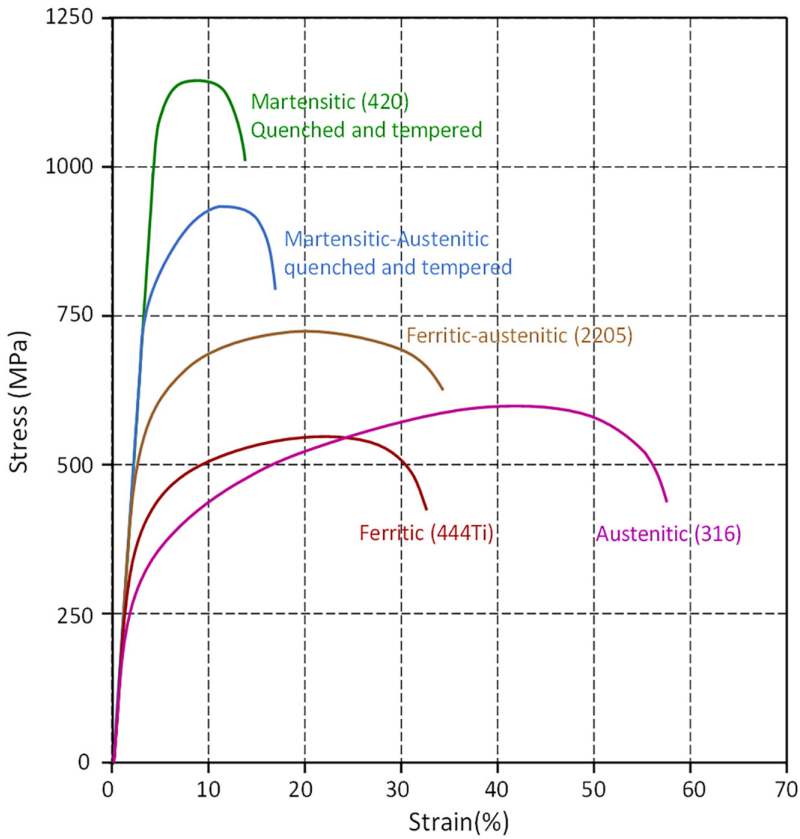

FIGURE 8.3 Stress-strain curves for different stainless steels at room temperature (Markeset *et al*. 2006).

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ductility required for use in concrete, stainless steels can be produced as ribbed bars. The relevant SSR has an elasticity modulus of around 200 kN/mm, which is comparable to carbon steel reinforcement (210 kN/mm). Duplex steels are of particular interest as reinforcement material because of their excellent mechanical properties in as-rolled conditions. For example, duplex steel grade 1.4462 (X2CrNiMoN 22-5) has a yield (proof) strength of 950 MPa, ultimate strength of 1059 MPa, and elongation (at break) of 14% for 10 mm bars when cold rolled (Markeset *et al.* 2006).

8.6.2 Grades and Standards

The main stainless steel rebar standards used nowadays are the BS 6744 and ASTM A955 standards. There are other standards such as XP135014 (France), SIA 262 (Switzerland), or DIN 488 (Germany), etc. Materials for structural stainless steel rebar and their required tensile properties are listed in Tables 8.7, 8.8 and 8.9.

TABLE 8.7
Stainless steel material grades in different standards (Stainless Rebar 2022)

Stainless steel type	BS 6744 EN grades	ASTM A955 UNS grades	ASTM A955 type
Austenitic with Mn	–	S24000	XM29
	–	S24100	XM28
Standard austenitic	1.4301	S30400	304
	1.4436	–	–
	–	S31603	316
	1.4429	S31653	316LN
Duplex	1.4162	–	–
	1.4362	–	–
	1.4462	S31803	–
	1.4501	–	–
Super austenitic	1.4529	–	–

TABLE 8.8
Tensile properties of different structural grades of BS 6744 (Stainless Rebar 2022)

BS 6744 Tensile properties	0.2% Proof strength $R_{p0.2}$ MPa min	Stress ratio $R_m/R_{p0.2}$ min	Elongation at fracture, % min	Total elongation at maximum force, % min
Grade 200	200	1.1	22	5
Grade 500	500	1.1	14	5
Grade 650	650	1.1	14	5

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TABLE 8.9
Tensile properties of different structural grades of ASTM A955 (Stainless Rebar 2022)

ASTM A955 Tensile properties	Grade 40 [280]	Grade 60 [420]	Grade 75 [520]
Yield strength, MPa [ksi] min	280 [40]	420 [60]	520 [75]
Tensile strength, MPa [ksi] min	500 [70]	620 [90]	690 [100]
Total elongation, % min	20	20	20

For critical structural components, stainless steel rebar is usually chosen. Using stainless steel and carbon steel in conjunction has no negative consequences.

8.6.3 Application at Extreme Temperatures

At high temperatures, especially above 500°C, carbon steel's proof stress drops dramatically. At temperatures below zero, it loses a lot of ductility and becomes more brittle. Most carbon steels are brittle below –20°C and would not be suitable for use as reinforcement in structures subjected to sudden impact loading or seismic events. For SSR, the situation is quite different. At much higher temperatures than carbon steel, austenitic stainless steels maintain their strength. Figure 8.4 depicts the variation of yield strength with temperature for various types of rebar, demonstrating the superior properties of stainless steel 1.4331 over ordinary carbon steel S500. As a result, such steels are more fire resistant and robust than carbon steel. Similarly,

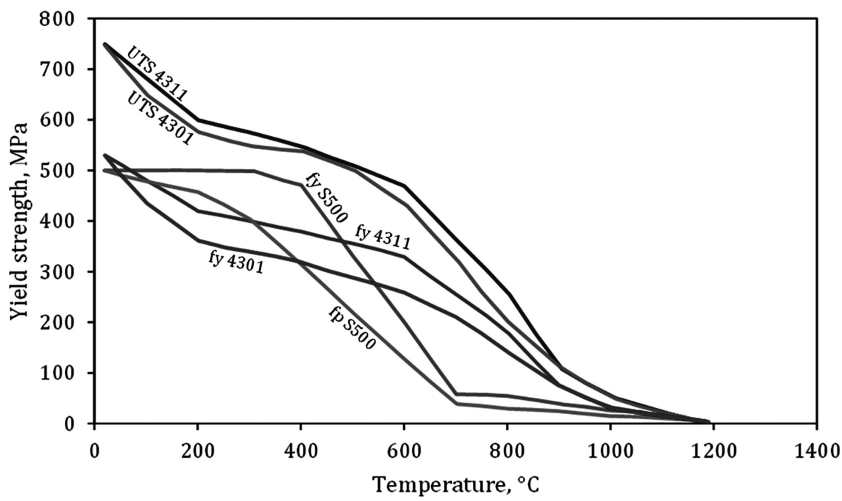

FIGURE 8.4 Variation of yield strength with temperature for different kinds of steel (Franssen and Gernay 2013).

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austenitic steels retain their strength and ductility at extremely low temperatures, known as cryogenic temperatures, which can reach –196°C. Furthermore, their strengths increase slightly as the temperature drops. Stainless steels are frequently used for their high temperature properties, even though they are most commonly used for their corrosion resistance. Stainless steels are used in applications that require high-temperature oxidation resistance and other applications that require high-temperature strength. The high chromium content of stainless steels, which is beneficial to their wet corrosion resistance, is also beneficial to their high-temperature strength and scaling resistance at elevated temperatures. Austenitic steels have a unique crystalline structure that allows them to maintain a high degree of toughness (Charpy impact test) over a wide temperature range, from above freezing to well below. Due to the crystalline structure difference between austenitic steel and duplex steel (where the duplex is partially ferritic), duplex steel loses a lot of toughness at low temperatures, starting around –50°C (Markeset *et al.* 2006).

8.6.4 Fatigue

When tested in the atmosphere, SSR has fatigue properties similar to carbon steel reinforcement. However, the unique situation of fatigue loading in a corrosive environment is a critical issue for reinforcement. In this case, the significantly higher corrosion resistance of SSR compared to carbon steel reinforcement results in significantly higher corrosion fatigue resistance of SSR. SSR's fatigue limit (the maximum amount of fatigue stress that can be sustained indefinitely) is determined by the steel's tensile strength. In this case, the higher strength of several types of SSR compared to carbon steel reinforcement results in a higher fatigue limit (Markeset *et al.* 2006).

8.7 APPLICATIONS

Even though the initial cost of stainless steel is significantly higher than that of carbon steel, its use can be justified on the basis that the increase in total project cost is small and is easily outweighed by the benefits of lower maintenance and repair costs, especially when disruption times and costs associated with such work are considered. Use corrosion-resistant reinforcement materials for the following applications:

a. Structures that are exposed to the attack of corrosion-promoting substances.
b. When the concrete cover and the concrete quality is – by design or otherwise – lower than the required values for the surrounding environmental conditions (e.g. in extremely slender elements).
c. When special structures, such as connections between precast and cast-in-place elements or heat-insulated joints between the structure and external structural elements, need to be constructed (e.g. balconies).
d. In prefabricated wall and roof elements where the reinforcement connects the outer and inner walls.

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e. When non-dense or dense lightweight concrete is designed to reach a required thermal insulation as well as low self weight.
f. In situations where access to the structure is severely restricted, making future inspection and maintenance expensive, such as with underground structures in aggressive soil.
g. In cases where maintenance is possible in future, but may cause extremely high indirect costs due to non-availability; for example, in bridges that are located on the main arterial highways in busy areas.

There are guidelines that can be followed in order to make the most out of the use of stainless steel reinforcement (Nürnberger 1996; Concrete Society 1998; Highway Agency 2002). The decision on which type of stainless steel to use depends on:

a. The degree of corrosion protection required.
b. Cost aspects.
c. Workability and application characteristics (mechanical and physical properties, weldability).

Offshore structures, piers at the sea coast, parts of highway structures subject to deicing salts or splash, multi-stored car parks, plants for the desalination of sea-water, concrete elements in thermal baths, and any type of repair work are all examples of typical applications that warrant the use of ferritic-austenitic and austenitic stainless steels due to lower maintenance costs. A guidance on location where use of stainless steel reinforcement is recommended in new highway structures is published by The Highway Agency (2002). It is possible to replace all of the carbon steel reinforcement on a structure with corrosion-resistant reinforcement; however, this would almost never be financially justifiable. It is recommended that replacement with stainless steel reinforcement be limited to those major components in which the consequences of future repair are likely to be highly disruptive and costly, as well as those in which the possibility of chloride attack is likely. Components of highway structures that may meet these requirements include:

a. Bridge decks carrying highly trafficked roads over busy railroad tracks with limited repair resources.
b. Exposed piers and columns in center reserves (but not deeply buried elements).
c. Deck slabs where maintenance access will be extremely difficult due to heavy traffic.

Utilizing stainless steel reinforcement in conjunction with carbon steel is an alternative method. It is not anticipated that stainless steel will replace a substantial portion of the massive tonnage of carbon steel reinforcements currently produced. Stainless steel applications should not be limited to chromium, nickel, or molybdenum steels with austenitic and ferritic-austenitic structures. Ferritic chromium-alloyed steels may be the best option in moderately aggressive environments, such as carbonated

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normal and lightweight concrete if chloride attack can be ruled out and the higher corrosion resistance of the more expensive stainless steels is not required.

8.8 EXAMPLES OF STAINLESS STEEL STRUCTURES

8.8.1 Hong Kong-Zhuhai-Macau Bridge

The Hong Kong-Zhuhai-Macau Bridge (HZMB) is a 55-kilometer-long bridge-tunnel system made up of three cable-stayed bridges, an underwater tunnel, and four artificial islands. A part of this bridge can be seen in Figure 8.5. The HZMB connects Hong Kong, Macau, and Zhuhai – three major cities in China's Pearl River Delta – across the Lingding and Jiuzhou channels. The bridge has an intended service life of 120 years. Because of the intended service life and the difficulty of repairing the bridge, duplex stainless steel grade EN 1.4362 was chosen. 1.4362 is a lean duplex stainless steel, and it was deemed appropriate both because of durability reasons and its cost efficiency (Jing and Fang 2017). In this bridge, stainless steel rebars are mostly used in splash zones, which are the most susceptible to chloride ingress (Dahlström and Persson 2018).

FIGURE 8.5 The Hong Kong-Zhuhai-Macau Bridge near Macau (Photo © N509FZ, Wikimedia Commons CC by SA 4.0, available at https://commons.wikimedia.org/wiki/File:West_section_of_Hong_Kong-Zhuhai-Macau_Bridge_(20180902174105).jpg, accessed 15 Sept. 2024).

8.8.2 The New Hastings Bridge in Minnesota

In Minnesota, the new Hastings Bridge (Figure 8.6) was constructed in 2013 to replace the functionally obsolete old truss bridge. Due to the approximately 34,000 vehicles per day that traverse the bridge, it is a vital connection. As a result, any disruption of traffic would be extremely costly, which led to the selection of stainless

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FIGURE 8.6 The new Hastings Bridge in Minnesota (Photo © Greg Gjerdingen from Willmar, USA, Wikimedia Commons CC by 2.0, available at https://commons.wikimedia.org/wiki/File:DSC00501_(29262177818).jpg, accessed 15 Sept. 2024).

steel as the optimal reinforcement material. Arminox UNS S32304 stainless steel was used for this project in order to ensure the bridge's long-term durability at a cost of approximately 120 million dollars. The manufacturer claimed at least 150 years of corrosion resistance. Arminox UNS S32304 is manufactured in the United States and in accordance with EN standards. It is a duplex stainless steel with the same composition as EN 1.4362. The concrete deck slab, end diaphragms, sidewalk, concrete end posts, and railings all contain approximately 300 tons of stainless steel (Dahlström and Persson 2018).

8.8.3 The Bridge over Winooski River in Vermont

In 2009, the Vermont Agency of Transportation replaced a 2.84 million dollar bridge that carries US Highway 2 over the Winooski River in East Montpelier. It was designed as a very low maintenance bridge. The superstructure has five weathering steel girders (no paint), with a bare concrete deck of high-performance concrete and solid stainless steel rebar. Stainless steel rebar does cost more initially, but the Vermont agency determined it to be worth the money over the life of the bridge. It was determined that overall cost escalation was only 3.7%. For the project, it was anticipated that a 975,000 dollar saving on maintenance and rehabilitation costs was achieved over the life of the project, compared to the 94,500 dollar initial increase in cost for stainless steel. Figure 8.7 shows a close-up photo of the stainless rebar in the deck of the bridge during the construction (Innovator 2012).

8.8.4 Saint George Bridge in Genova

The stainless steel bars with improved adhesion play a critical role in the new Ponte San Giorgio in Genova (Figure 8.8), designed by Renzo Piano, which was inaugurated on August 3, 2020. The stainless rebars ensure not only structural strength, but also resistance to corrosive phenomena and maximum structural safety (Stainless Rebar 2022).

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FIGURE 8.7 Solid stainless steel rebar on a bridge deck before casting (Photo © FHWA, Wikimedia Commons CC by 2.0, available at https://commons.wikimedia.org/wiki/File:Deck_Reinforcement_VT_(8495188335).jpg, accessed 25 Sept. 2024).

FIGURE 8.8 Saint George Bridge in Genova [Source: Inossidabile magazine n° 220, edited by Centro Inox Servizi Srl. Stainless steel rebar produced and furnished by Acciaierie Valbruna. Reproduced with permission].

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8.8.5 The Haynes Inlet Slough Bridge in Oregon

In 2004, a stainless steel reinforced concrete bridge carrying US Route 101 over Haynes Inlet was inaugurated in Oregon, United States. The bridge is designed to have a lifespan of 120 years, which is nearly 2.5 times that of the bridge it would replace (Emerald Insight 2002). The estimated cost of the bridge was 12.5 million dollars. It was estimated that using stainless steel rebars would save 25 million dollars by eliminating the need to replace or repair the bridge in 50 years. The Haynes Inlet Bridge utilized nearly 400 tons of stainless steel rebars, more than any other North American bridge at the time. Due to the harsh environment the bridge was exposed to, including chloride ingress and seismic activity, stainless steel of the highest quality was essential. Therefore, a duplex stainless steel Alloy 2205 was used. 316 LN, equivalent to EN 1.4429, was the most common stainless steel used for bridges at the time. Alloy 2205, which is equivalent to EN 1.4462 and is a duplex steel with a mixture of austenitic and ferritic microstructures, was chosen instead due to higher requirements for strength and durability. The bridge was designed with stainless steel reinforcing bars in the most corrosive areas and carbon steel reinforcing bars in substructure elements not directly exposed to corrosion (Dahlström and Persson 2018).

8.9 COST-EFFECTIVENESS OF STAINLESS STEEL

Stainless steel rebar is, undoubtedly, an expensive material compared to its carbon steel counterpart. Generally, the cost of SSR is about 7 to 9 times higher than that of common mild steel rebar. This high cost gives a general perception that structures with SSR will be very costly, thus, discouraging engineers and owners from considering SSR for construction. However, such a perception may not be fully correct. If the long-term life cycle cost (LCC) is taken into account, the scenario changes significantly. It may even be found that if the total cost of a structure, which includes the cost of construction and long-term maintenance and repair, is taken into account, the cost with SSR may even be lower than the same with common carbon steel counterpart. The following articles discuss some of the projects and studies depicting that structures with SSR may be cost-effective as well.

8.9.1 Study of CIMbéton (2004)

CIMbéton (2004) conducted a study in 2004 on the relative cost of using stainless steel rebar for a typical road bridge construction. An estimate of the overall bridge cost was calculated for different amounts of stainless steel replacing the ordinary mild steel rebar. The summarized result is given in Table 8.10. Grade 304 stainless steel was considered. Market prices and grade selection would affect the figures. Nonetheless, the outcomes are consistent with typical road bridge construction. The study concluded that a simple price comparison of stainless steel vs. carbon steel with identical implementation costs is insufficient. Stainless steel makes structures lighter and allows for better sizing and material usage. The cost of replacing steel

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FIGURE 8.9 The Haynes Inlet Slough bridge in Oregon, with Alloy 2205 stainless steel bars in critical areas (©Michael Goff, Oregon DoT, reproduced with permission).

with stainless steel, whether partially or completely, is typically lower than our perception, and is quickly offset by significant savings in maintenance and repair costs, as well as the cost of keeping the facility closed during the repair period.

TABLE 8.10
Construction cost of a road bridge for different amounts of SSR.

Percent of mild steel replaced by stainless steel rebar	Total bridge cost indications, %
0%	100
10%	103
25%	107.5
50%	115
100%	130

8.9.2 Study of Schnell and Bergmann (2008)

Schnell and Bergmann (2008) conducted a study in the aftermath of the I-35W Bridge collapse in Minneapolis in 2008 to raise awareness of the use of solid stainless steel

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reinforcing bar as a high-strength, corrosion-resistant alternative rebar product. While solid stainless steel reinforcing bar can increase the cost of the bridge deck by up to 12% (when compared to carbon steel reinforcing), they claim that the economic value of the longer life outweighs the initial higher cost. In most cases, the extra cost of solid stainless steel reinforcing bar is less than 1.5% of the overall structure cost. They conducted a systematic study to compare the total cost of a bridge that includes the cost of construction and long-term life cycle cost. Their study included different alternatives such as using epoxy coated rebar (ECR), high performance concrete (HPC), galvanized rebar, micro-composite multi-structural formable (MMFX2) steel, fiber reinforced plastic (FRP) rebar, stainless steel clad rebar, and solid stainless steel rebar including proprietary CarTech EnduraMet® 32 stainless steel. They considered a bridge with ECR as the base structure. A summarized result of their study is shown in Table 8.11.

Schnell and Bergmann (2008) included three brief case studies in their research. They concluded that using more expensive and long-lasting materials may not always increase the project's initial cost. Savings elsewhere will completely offset the increased cost of stainless steel reinforcing. The owner and the general public benefit from the stainless reinforcing's longer lifespan.

TABLE 8.11

Comparison of initial cost and life cycle costs of new bridges with various types of deck reinforcing.

Reinforcing type	ECR, galvanized	MMFX2	FRP	Solid stainless	EnduraMet© 32 stainless
Deck cost (compared to total initial cost of "base" structure)	38.00%	39.14%	39.90%	42.18%	39.90%
Steel cost (compared to total initial cost of "base" structure)	31.00%	30.50%	30.50%	30.50%	31.00%
Foundation cost (compared to total initial cost of "base" structure)	25.00%	25.00%	24.00%	24.00%	24.00%
Earthwork, etc. cost (compared to total initial cost of "base" structure)	6.00%	6.00%	6.00%	6.00%	6.00%
Total initial cost of structure	100.00%	101.14%	100.40%	102.68%	100.40%
Estimated life (years)	40	50	65	100	100
Present worth of deck replacement at end of life	9.89%	6.88%	3.93%	1.05%	1.00%
100-year life cycle cost as a percentage of initial cost of the "base" structure	111.48%	108.02%	103.88%	103.74%	101.40%

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8.9.3 Study on the Schaffhausen Bridge

In 1995, the Schaffhausen Bridge (Figure 8.10) was completed across the Rhine. The bridge is designed to withstand atmospheric corrosion as well as de-icing salts. The bridge is expected to last for 80 years without deteriorating structurally or requiring significant maintenance. The study was conducted by McGurn (1998). Three rebar reinforcement options were considered:

- Carbon steel would have required costly repairs every 25 years, causing significant traffic disruption.
- Epoxy-coated carbon steel: less extensive repairs would be required every 25 years, resulting in some traffic disruption.
- Carbon steel in critical areas with stainless steel in other areas: meets design life requirements without any significant repairs.

The comparative costs of the three alternatives are shown in Table 8.12. It has been shown that proper use of stainless steel leads to an extra initial cost of only 0.5%, paid back 30 times by the savings achieved over the lifetime of the bridge.

FIGURE 8.10 The Schaffhausen bridge across the river Rhine, 1995 (Photo © Хрюша, Wikimedia Commons CC by SA 3.0, available at https://commons.wikimedia.org/wiki/File:Rheinbruecke_N4_06_10.jpg, accessed 15 Sept. 2024).

8.9.4 The Progreso Pier, Mexico

The Progreso Pier (Figure 8.11) is the longest pier in the world which was built in 1941 and used stainless steel as the concrete reinforcement. Life cycle assessment (LCA) and life cycle cost (LCC) analyses were conducted by Santero *et al.* (2013) for this pier. A comparative assessment was done by them with a hypothetical alternative of carbon steel rebar. They calculated both the global warming potential (GWP) and the LCC over a period of about 80 years. A summary of their findings is depicted

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TABLE 8.12
Life cycle cost summary of the Schaffhausen Bridge (in US dollars) (McGurn 1998)

Description	Carbon steel	Epoxy coated rebar	Stainless steel
Material costs	5653	21,670	61,138
Fabrication costs	0	0	0
Other installation costs	10,766,918	10,766,918	10,766,918
Initial costs	10,772,571	10,788,588	10,828,055
Maintenance	0	0	0
Replacement	176,724	53,017	–97
Lost production	1,530,083	1,530,083	0
Material related	0	0	0
Operating costs	1,706,807	1,583,100	–97
Total LCC	12,479,378	12,371,688	10,827,958
Increase w.r.t. initial cost with carbon steel	15.85%	14.84%	0.5%

FIGURE 8.11 The Progreso Pier, Mexico (Photo © Minerva Castro, Wikimedia Commons CC by SA 4.0, available at https://commons.wikimedia.org/wiki/File:Puente-PtoProgreso.jpg, accessed 15 Sept. 2024).

in Figures 8.12 and 8.13. It can be observed that the global warming potential (GWP) with carbon steel rebar is much higher than the same with stainless steel for a life expectancy of more than 50 years. The life cycle cost analysis shows that the net present cost of structure with carbon steel rebar becomes much higher after the age of 50 years with a 0.01% discount rate. Even at a much higher 4% discount rate, the net present cost is at par with the hypothetical carbon steel counterpart.

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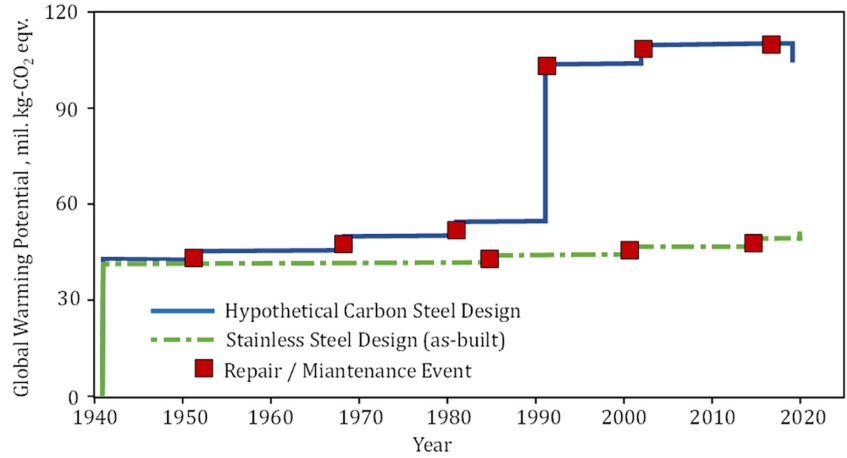

FIGURE 8.12 Comparison of GWP over 79-year analysis period (Santero *et al.* 2013).

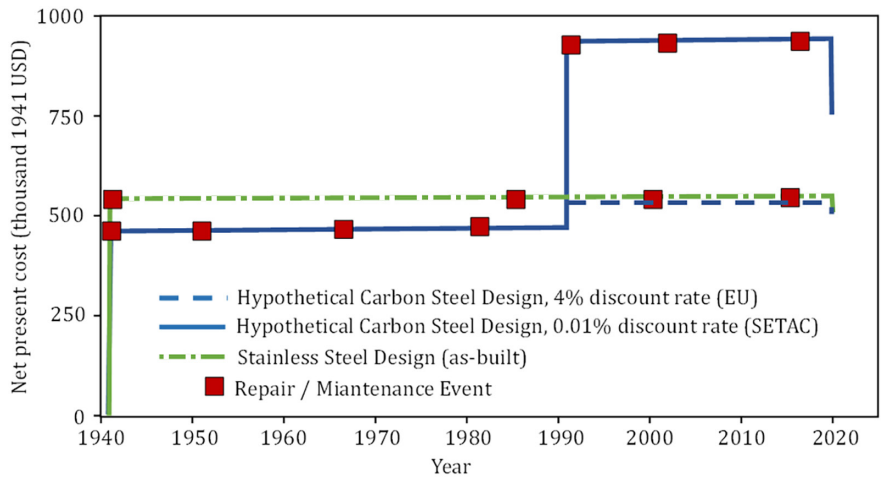

FIGURE 8.13 Life cycle costing results (Santero *et al.* 2013).

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QUESTIONS

8-1. What is the basic objective of using stainless steel in concrete structures instead of conventional carbon steel? Discuss. Write some advantages of using stainless steel rebar.

8-2. Discuss the comparative advantages and drawbacks of various approaches to prevent concrete deterioration due to rebar corrosion.

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8-3. Provide a brief description of different types of stainless steel alloys including a tabular comparison of characteristics.
8-4. Describe the following classification systems for stainless steel:
   a. Traditional classification system.
   b. European classification system.
   c. Unified numbering system.
8-5. Describe different physical properties of stainless steel.
8-6. Explain, with illustrations, the stress-strain behavior of different types of stainless steel.
8-7. Discuss the effects of temperature on mechanical properties of different types of stainless steel.
8-8. Discuss the possible applications of stainless steel as reinforcement in concrete structures.
8-9. Discuss the cost-effectiveness of stainless steel reinforcement in concrete structures with the help of at least two past studies.

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9 Coated Reinforcing Steel

9.1 INTRODUCTION

Various types of corrosion-protection systems are used for reinforced concrete structures. One direct approach to reduce the risk of corrosion of the reinforcing steel that results in concrete damage, even if salt penetrates the concrete, is to coat the reinforcement with a non-metallic impervious material such as epoxy, or with a metallic coating such as zinc (galvanizing). Because this book is concerned with reinforcing steel, this chapter only deals with coated reinforcing steel as a corrosion-protection system for reinforced concrete structures.

Coated reinforcing steel, such as zinc- or epoxy-coated rebar shown in Figure 9.1 is intended to resist corrosion and delay deterioration of reinforced concrete structures. Thus, construction activities have to be carried out at the job site with the overall objective of maintaining the integrity and quality of the factory-produced, coated reinforcing steel. Although the recommended practices are relatively simple, they are effective ways to minimize damaged coating that should extend the corrosion protection benefit.

When a project requires the use of coated reinforcing steel, the pre-construction meeting is an ideal time to initiate the steps for maintaining and achieving the quality of the coated reinforcing steel. Effective communication at that time will minimize future problems and disputes. Applicable provisions in the project specifications should be reviewed. The owner's representative should clearly communicate the intent to enforce those provisions. All parties involved with the coated reinforcing steel for the construction project, including those directly responsible for the reinforcement and concrete operations, should participate in the meeting. The roles and responsibilities of the parties should be established or clarified (CRSI 2011). Specific construction practices and inspection procedures should be reviewed. Anticipated delivery schedules for coated reinforcing steel and proposed storage sites should also be outlined.

9.2 HISTORY OF COATED REINFORCEMENT

Corrosion of bridge decks was not a major concern throughout most of the 1900s. Corrosion-related damage had been uncommon on properly designed and constructed bridges. This changed substantially in the 1950s, when highway agencies began using deicing salts (e.g. sodium chloride) to clear snow and ice from highways and bridges. This "bare pavement" approach made roads safer for drivers, but it also resulted in a significant rise in deterioration due to chloride penetration on highways and bridges.

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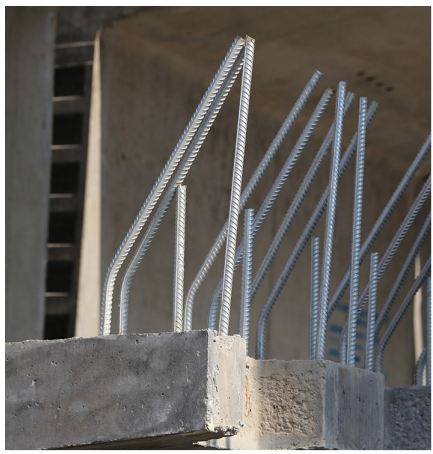
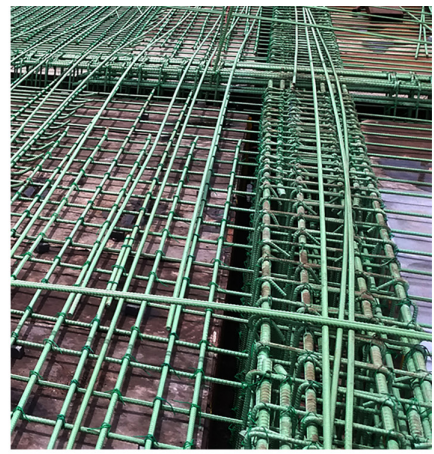
Galvanizing, or coating a steel product with a thin layer of zinc (Zn), to protect steel from corrosion had been in practice for more than a century. The first standard specification for hot-dip galvanizing was published by ASTM in 1927 and the US Navy used galvanized rebar in a bridge in Bermuda in 1953 (Lane and Kleinhans 2016). It is an established means of protecting steel reinforcement and other embedded steel components against corrosion in concrete. It has been quite widely used for more than 100 years in many places for a variety of aggressive and mild-to-moderately corrosive environments for concrete structures (Figure 9.1a).

Compared to galvanizing, epoxy coating of reinforcement for corrosion protection is a relatively new technology that emerged in the 1970s. In 1973, a bridge in West Conshohocken, Pennsylvania, was the first to use epoxy-coated reinforcing steel. The epoxy-coated steel reinforcement bars were used on four of the bridge's spans. The National Institute of Standards and Technology (NIST) conducted testing on a variety of coatings to assess their corrosion-protective properties, chemical and physical durability, and chloride permeability characteristics (Clifton *et al.* 1975). Based on the results of the tests, a fusion-bonded epoxy coating applied to reinforcing steel was offered as a means to improve bridge deck corrosion resistance (Figure 9.1b).



(a) Galvanized rebar in side barrier of a bridge



(b) Epoxy coated rebar on jetty deck
(Photo courtesy - BSRM)

FIGURE 9.1 Examples of zinc and epoxy-coated rebar.

9.3 STANDARD COATING FOR REINFORCEMENT

ASTM recognizes the following types of coated bars, wire, and welded-wire reinforcement.

a. Epoxy-coated (ASTM A775, A884, A934).
b. Galvanized (ASTM A767).
c. Dual-coated (ASTM A1055).
d. Vinyl-coated (ASTM A933).

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In North America, epoxy-coated reinforcing (ECR) steel is an extensively utilized coated bar. The majority of ECR manufacturing plants in North America are certified by CRSI, which conducts random inspections of facilities to guarantee that they have the processes, personnel, and equipment necessary to create high-quality commodities. Over the last 40 years, ASTM requirements have evolved to include thicker coatings, as well as suitable surface cleanliness and roughness qualities.

Vinyl-coated bars are coated similarly to epoxy-coated bars, with the exception that the coating substance is a vinyl polymer. However, these bars have not been widely utilized in construction.

Dual-coated bars are produced in a way similar to that of epoxy-coated reinforcing steel by spraying prepared reinforcing steel with a zinc alloy and then coating the bars with epoxy. Several agencies, notably Florida and Vermont DOT, have elected to incorporate these bars in their standard specifications. In other states, demonstration projects are also being carried out (McDonald 2014).

To produce galvanized reinforcing steel, appropriately prepared reinforcing bars are immersed in molten zinc, which is commonly termed as hot dip galvanizing (HDG). The final product is composed of several zinc-iron and zinc layers. Several plants outside of North America use another technique that continuously applies zinc alloy coatings to reinforcing steel, and an ASTM specification for this process is now being developed.

Although these are not commercially commonplace, stainless steel-clad reinforcing bars have been used in experimental concrete buildings. Ceramic-coated reinforcing bars have also been studied in the lab, although they haven't been applied to concrete constructions yet.

A *Specialty and Corrosion-Resistant Product Guide*, published by CRSI, is a helpful resource for engineers on available reinforcing materials.

9.4 REBAR COATING METHODS

A passive layer generated by the alkaline atmosphere of fresh concrete protects steel rebar in concrete constructions. The intrusion of chloride or carbonation of the concrete cover layer, on the other hand, can dissolve this protective film. Corrosion begins when metal is corroded and exposed to oxygen and moisture. Corrosion products have a volume that is 2 to 6 times that of the original steel they consume, causing bursting or splitting tensile stress in the surrounding concrete. When the stress exceeds the concrete's tensile strength, the concrete cover cracks or spalls. Meanwhile, corrosion affects the strength of the steel-concrete bond and the cross-sectional area of the steel rebar (Tang 2016). As a result, one of the most common causes of premature deterioration in reinforced concrete structures is steel reinforcement corrosion.

The use of corrosion inhibitors or high-performance admixtures, protective coatings or corrosion-resistant bars etc., are just some of the many methods or techniques that have been developed to protect steel rebar from corrosion in concrete structures (Tang 2016). Because protective coatings can create a physical and chemical barrier between steel rebar and the corrosive environment, it is one of the most effective

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and efficient approaches. In this chapter, protective coatings that are commonly used to prevent steel rebar from corroding in concrete structures are presented, with the primary focus being:

a. Fusion-bonded epoxy (FBE).
b. Hot-dip galvanizing (HDG).

Details of these methods are discussed in the following sections. Making sure that the concrete covering the reinforcement is thick enough and that the concrete itself is dense and impermeable is the most economical and effective way to reduce the risk of rebar corrosion. It must be noted that these coating techniques are not, in any way, to be regarded as a replacement for sound concrete construction practice.

9.5 EPOXY-COATED REBAR

The Concrete Reinforcing Steel Institute (CRSI) reports that epoxy-coated reinforcing steel was initially utilized in the construction of a bridge in the state of Pennsylvania in the year 1973. Earlier, there was less concern about the possibility of automobiles sliding on icy roads or clearing the roads of ice and snow during the winter by application of salts or de-icing chemicals which is the source of chlorides. The use of deicing chemicals to clear bridges and highways did not begin until the 1950s and the consequences started becoming apparent in the 1960s. Unfortunately, the usage of these chemicals caused repair prices to increase due to corrosion issues. A more effective solution to lessen the impact of the problem became essential, and after testing a variety of protective coatings, fusion-bonded epoxy coating of rebar (Figure 9.2) proved to be a suitable alternative. More on epoxy material can be found in Section 10.7.1.

FIGURE 9.2 Fusion-bonded epoxy-coated reinforced rebar (Photo courtesy – BSRM).

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9.5.1 Fusion-Bonded Epoxy (FBE) Material

In the fusion-bonded epoxy coating process, epoxy is applied in a powdered form which is a mixture of resin, hardener, and other ingredients. The components of a powder coating are:

- Resin.
- Hardener or curing agent.
- Fillers and extenders.
- Color pigments.

The resin used in fusion-bonded epoxy coatings is a resin of the "epoxy" type. The resin molecule has an "epoxy" or "oxirane" structure, which is made up of a three-membered cyclic ring with one oxygen atom joined to two carbon atoms. Bisphenol-A and epichlorohydrin derivatives are the most widely used FBE resins. To achieve different properties, combinations, or additions, FBE formulations frequently use other types of resins (such as bisphenol F type). Resins come in a variety of molecular lengths to give the finished coating special qualities.

The curing agent or hardener is the second-most significant component of FBE coatings. Along the molecular chain of the epoxy, curing agents either interact with the epoxy ring or the hydroxyl groups. In the production of FBE, a variety of curing agents are employed, such as dicyandiamide, aromatic amines, aliphatic diamines, etc. The final FBE coating's characteristics, such as its cross-linking density, chemical resistance, brittleness, flexibility, etc. are determined by the curing agent. Their relative equivalent weights determine the proportion of epoxy resins to curing agents in a formulation.

In order to achieve the desired properties, FBE coatings also contain fillers, pigments, extenders, and other additives. These elements regulate properties like permeability, hardness, color, thickness, gouge resistance, and more. All of these ingredients are typically dry solids and do not react when they are in dry powdered form.

The microstructure of an FBE coating on a steel substrate is depicted in Figure 9.3. Due to chemical reactions that occur during the fusion process, some air bubbles are present around the steel.

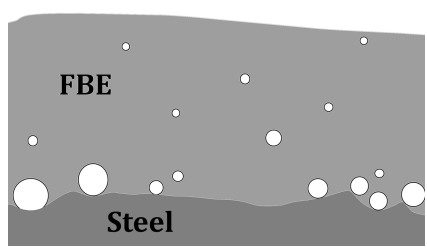

FIGURE 9.3 Microstructure of epoxy coating on rebar (adapted from Tang 2016).

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9.5.2 Manufacturing of Fusion-Bonded Epoxy-Coated Rebar

The manufacturing process of fusion-bonded epoxy (FBE) coated rebars involves four important steps – cleaning and surface preparation, induction preheating, fusion with epoxy powder application, and curing. Coating operation involves moving the straight length of the rebar through the application process on a moving conveyor. Specialized coating operation units may use a special hanger system. The general steps are as follows.

- **Degreasing** – Surface contaminants such as oil, grease, etc. are removed from rebar by solvent cleaning or by burning.
- **Surface preparation** – Using abrasive grit or shot blasting, reinforcement bars are cleaned to a white or near-white metal finish. This removes contaminants such as mill scale and rust from the steel. Additionally, the surface is textured to create an anchor profile. Salt contamination is eliminated during this procedure.
- **Heating** – Using electrical induction heaters, reinforcement bars are heated to temperatures ranging from 200 to 240°C. In some installations, however, natural gas is also used for heating.
- **Powder application** – The heated steel is moved through a powder spray booth where a number of spray nozzles apply dry epoxy powder. As the powder leaves the spray gun, the particles acquire an electrical charge. These electrically charged particles are drawn to the grounded steel surface, resulting in an even coating application. When the dry powder comes into contact with the heated steel, it melts and flows into the anchor profile (i.e. the microscopic peaks and valleys on the surface) and covers the ribs and deformations of the bar and bonds with the steel. A chemical reaction is also triggered by heat, resulting in the formation of complex cross-linked polymers from the powder molecules, which are responsible for the material's advantageous properties. Typically, coating thicknesses in the range of 50 to 450 micrometers are obtained, although thinner or thicker ranges may be specified based on service conditions.
- **Curing and cooling** – When the "gel time" has elapsed, which is typically a few seconds after coating application, the molten powder solidifies into a coating. Under hot conditions, the resin component of the coating undergoes cross-linking, which is known as "curing." Either the residual heat on the steel or additional heating sources are required to achieve complete curing. In the case of long-cure FBEs, a full cure can be achieved in a few minutes, depending on the FBE coating system. During the curing period, the coating becomes solid. In some plants, curing is frequently followed by an air or water quench that quickly lowers the bar's temperature to make testing and handling easier.
- **Inspection and testing** – FBE coated reinforcement bars are put through testing at the coating facility, following the specifications mentioned in the applicable standard. Coating continuity and thickness checks are performed

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both online and offline. Bending the bar is another frequent method of testing the adhesion of coated bars. In addition to this, numerous other tests are carried out in a lab for every batch, including tests for chemical resistance, short-term spray resistance, resistance to continuous boiling water, abrasion resistance, and impact resistance.

9.5.3 Testing of FBE Coated Rebar for Quality Control

Requirements for epoxy-coated steel reinforcing bars are clearly outlined in ASTM A775 specification. These are briefly discussed below.

9.5.3.1 Coating Thickness

After curing, the coating thickness measurements should be 175 to 300 microns for bar sizes (metric) 10 to 16, and 175 to 400 microns for bar sizes 19 to 57. The upper thickness limit shall not apply to areas of damaged coating that have been repaired. ASTM A775 specifies the procedure for measuring coating thickness.

9.5.3.2 Coating Continuity

The objective of this test is to determine the number of holidays or such other defects on the coating. Here, *holiday* is a discontinuity in a coating that is not discernible to a person with normal or corrected vision. On average, there shall not be more than three holidays per meter on a coated steel reinforcing bar. The average applies to the entire length of a bar as it is produced.

9.5.3.3 Coating Flexibility

Epoxy coating on rebar must be flexible and stretchable enough to withstand bending without any cracking or disbonding. This is tested by subjecting an epoxy-coated bar to being bent around a mandrel of a specified diameter. For bar designation #3 to #11, the mandrel diameter is 8 times the bar diameter, and the bend angle shall be 180°. For #14 and #18 bars, the mandrel diameter is 10 times the bar diameter, and the bend angle is 90°. For #3 to #6 bars, the bending operation shall be completed within 15 seconds. For higher diameter bars, the bending operation has to be completed within 45 seconds. The bend operation shall be such that the two longitudinal ribs are placed in a plane perpendicular to the mandrel radius. The temperature of test specimens must be between 20° and 30°C. Cracking or disbonding of the coating visible to a person with normal or corrected vision on the outside radius of the bent bar shall be considered a cause for rejection of the coated steel reinforcing bars.

9.5.3.4 Chemical Resistance

A chemical resistance test is intended for evaluating the resistance of epoxy coating materials when exposed to various concentrations of reagents or suspected contaminants. The test is conducted in compliance with the ASTM G20 test methodology.

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9.5.3.5 Cathodic Disbondment

This test covers accelerated procedures for simultaneously determining comparative characteristics of epoxy coating systems applied to steel reinforcement for the purpose of preventing or mitigating corrosion that may occur where the coated rebar will be in concrete and may or may not receive cathodic protection. Details of the test procedure are outlined in the ASTM test procedure G8.

9.5.3.6 Salt Spray Resistance

The salt spray test is another method to evaluate the corrosion resistance of epoxy coatings which is covered in ASTM B117. In this test method, the coating to be tested is exposed to continuous salt spray for a pre-determined period of time. Samples are exposed to an atomized spray of 5% NaCl solution at 35°C and then placed in a humid cabinet for about 33 days. Each specimen shall contain three intentional 3 mm diameter drill holes as defects. The average coating disbondment radius of nine test sites on three coated steel reinforcing bars shall not exceed 3 mm when measured from the edge of the intentional coating defect. Further details of the test procedure are described in ASTM B117.

9.5.3.7 Chloride Permeability

This test is performed to evaluate the chloride permeability of the coating film. The test is conducted on a specially prepared film of epoxy coating having a thickness of about 200 m microns. Before being installed in the cell, the films chosen for testing must be handled with care and inspected for flaws. The cell shall be comprised of two glass compartments separated by a coating film sandwiched between two glass plates, each with a central 25 mm hole. 175 mL of 3*M* NaCl shall be put in one compartment, and 115 mL of distilled water shall be put in the other. Utilizing a specific ion meter, the activity of chloride ions passing through the film shall be determined. The test shall continue for 45 days. The accumulative concentration of chloride ions permeating through the film shall be less than $1 \times 10^{-4}M$.

9.5.3.8 Relative Bond Strength in Concrete

The relative bond strength of the steel reinforcing bars to concrete shall be determined with beam-end specimens by the method described in Test Method A944 using #19 (metric) steel reinforcing bars with a relative rib area between 0.075 and 0.085. Three to six coated bar specimens and three to six uncoated bar specimens shall be tested. The mean bond strength of the coated bars shall not be less than 85% of the mean bond strength of the uncoated bars. Further details of the testing procedure can be found in ASTM A944.

9.5.3.9 Abrasion Resistance

Abrasion resistance of epoxy coating is carried out in accordance with ASTM D4060 on steel panels coated with epoxy. A Taber Abraser apparatus is used with a CS-10 wheel of 1.0 kg load per wheel. After testing, the weight loss shall not exceed 100 mg per 1000 cycles.

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9.5.3.10 Impact Test

The resistance of the steel reinforcing bar coating to mechanical damage shall be determined by the falling weight test. A testing arrangement similar to that described in ASTM Test Method G14 shall be used along with a 1.8 kg tup having a nose diameter of 16 mm. The impact shall occur on the low-lying areas on the coated steel reinforcing bars, that is, between deformations or ribs. The test shall be performed at 24 ± 2°C. With an impact of 9 N-m, no shattering, cracking, or bond loss of the coating shall occur except at the impact area, that is, the area permanently deformed by the tup.

9.5.4 Material Specification

When project specifications include the use of epoxy-coated rebar, project owners must follow the relevant ASTM standards. ASTM A775, *Standard Specification for Epoxy-Coated Reinforcing Bars*, and ASTM A934, *Standard Specification for Epoxy-Coated Prefabricated Steel Reinforcing Bars*, are the most extensively used standards for epoxy-coated rebar. One word defines the difference between these two standards: prefabricated. ASTM A775 covers lengths of fusion-bonded epoxy-coated bar, whereas ASTM A934 covers fusion-bonded epoxy-coated bars that are cut and bent into specified shapes, sizes, and lengths, such as stirrups and hooks. This takes place prior to cleaning and powder coating application (Cutler 2015).

9.5.5 Handling, Placing, and Storage

The coating on epoxy-coated reinforcing bars should be protected as much as possible throughout construction operations. Simply said, handling, storing, and positioning epoxy-coated bars requires caution (CRSI 2011). Appendix Xl of ASTM A775 and A934 provides detailed standards for the handling and placement of epoxy-coated bars, whereas D3963 provides specific criteria for the manufacture and placement of epoxy-coated bars. In addition, ACI 301 specifies standards for treating epoxy-coated reinforcement comparable to those found in ASTM regulations. Such documentation or alternative procedures should be expressly referenced in the project requirements, and it is advised that these materials be reviewed prior to bar installation.

9.5.5.1 Handling During Manufacturing and Shipment

Small coating damage may occur during the manufacture and handling of epoxy-coated bars prior to shipment to the construction site. The two ASTM requirements for epoxy-coated bars (ASTM A775 and A934) stipulate that all damaged coating areas must be repaired (touched up) prior to shipment of the coated reinforcing bars to the job site. Epoxy-coated reinforcing bars should not be expected to be fully free of coating damage; nonetheless, all coating damage caused by shipping, handling, and placement must be repaired. The project standards should limit the amount of damage that can be fixed to less than 2% of the coated bar's surface area per foot. The damaged coating must be restored (patched) onsite using patching material

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supplied by the epoxy coating manufacturer. The patching material must be applied per the manufacturer's published directions. Before casting concrete over the coated bars, the patched portions must cure, which can take up to eight hours. Field patching can be a time-consuming and costly process. It is more efficient and cost-effective to handle and position coated reinforcing bars with care than to repair substantial coating damage. If the coating damage (which must all be fixed) exceeds the criteria for the project, the coated bars will be susceptible to rejection and will need to be replaced, which could create construction delays.

9.5.5.2 Handling and Storing at Site

Upon the arrival of epoxy-coated reinforcing bars at the construction site, the amounts of bars must be thoroughly compared to the bill of materials. Patching material (touch-up) kits and instructions on how to use them may be included with the shipment of coated bars. During the unloading process, the coated bars should be inspected for shipping-related coating damage. If broken or missing bars are discovered, the carrier must be contacted. Bars having substantial coating damage that cannot be fixed onsite will need a supply of newly coated bars.

When unloading epoxy-coated bars from a truck, care must be taken to reduce bundle scraping and bar-to-bar abrasion caused by sags in the bundles. The contact regions of the equipment used to handle the coated bars should be protected. Utilize nylon slings or padded wire rope slings. Coated bar bundles should be lifted from multiple locations. Hoisting coated bar bundles with a spreader beam or equivalent device is an excellent approach for preventing sags. Coated bars should be stored on protected cribbing, such as wooden or padded cribbing, above the ground. The supports should be close enough together to prevent the bundles from sagging. If it is required to stack bundles of straight coated bars, appropriate blocking should be placed between the bundles. To reduce the number of handling activities, reinforcing bars, particularly coated bars, should be stored near the location where they will be installed in the structure. Coated bars and coated bar bundles should not be dropped or dragged. The hues of the coatings vary based on the pigments utilized in the epoxy powder. When coated bars are kept outside and exposed to sunshine for an extended amount of time, the coating's color may fade. The coating is unaffected by color fading, so the bars should not be rejected on this basis.

Coated bar deliveries to the job site should be scheduled and coordinated to eliminate the requirement for long-term storage. It is not advisable to store coated bars from one construction season to the next. If circumstances or other situations necessitate storing epoxy-coated bars outside, the bars must be shielded from sunlight (as ultraviolet radiation destroys the coating over time) and salt spray and kept out of weather effects. CRSI recommends exposing uncoated bars to ultraviolet light for no more than 30 days (Cutler 2015). Epoxy-coated rebar must be stored separately from non-epoxy-coated rebar, and surface damage should be avoided using protective measures such as storage on wooden racks or plastic or rubber-coated steel racks. The project specifications should contain criteria for storage on the job site, which are often derived from relevant ASTM or ACI specifications. Coated bars or bundles should be covered with opaque polyethylene sheeting or another suitable protective

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material that is securely fastened and provides air circulation around the coated bars to prevent condensation under the protective covering.

9.5.5.3 Handling During Placement

The placement of epoxy-coated reinforcing bars is similar to the placement of uncoated bars, and the techniques described in standard literature apply. Prior to placement in formwork, bars with damaged coatings must be restored with an approved repair material and technique. Coated bars require additional handling and placement precautions in order to prevent damage. During all placement activities, the same hoisting and handling methods and precautions should be employed and adhered to as when unloading epoxy-coated bars from a truck, as stated previously. Once bundles of coated bars have been separated, it is important to avoid dragging one bar over another or across any rough surface. Taking safeguards based on common sense should limit coating damage during placement procedures. With such sensible considerations, the repair of damaged coating or field touch-up is typically minimal, according to past experience.

Typically, epoxy-coated reinforcing bars require longer tension lap splices than uncoated bars. The lap lengths will be depicted on the positioning drawings. For mechanical splices, the project specifications should stipulate that all parts of the splices, including steel coupling sleeves, bolts, and nuts, must be coated with the same patching material used to repair damaged coating on the bars; and damaged coating on the bars in the vicinity of the splices must be repaired. For welded splices, the welds and steel splice components used to splice bars must be covered with the same patching material used to repair broken epoxy coating on the bars; damaged coating on the bars near the welded splices must be fixed. When welding coated bars, sufficient ventilation must be provided.

If epoxy-coated reinforcing bars must be cut on the job site, the cut ends must be coated (touched up) using the same material used to repair damaged coating. Flame-cutting epoxy-coated bars is not permitted.

Additionally, the project specifications should include any particular requirements for field bending or straightening of epoxy-coated, partially implanted, hardened concrete bars. The project specifications should stipulate that coating damage must be repaired following bending or straightening. If the engineer allows the use of heat for field bending, there must be enough ventilation. Bending or straightening of epoxy-coated ASTM A934-compliant bars in the field should not be permitted.

After epoxy-coated reinforcing bars have been installed, workers should avoid walking on them as much as possible. The coated bars in place should be prevented from being damaged by dropped tools or other heavy construction items. Coated bars should not be harmed by the concrete carrying and laying equipment. Runways for concrete buggies and hoses for pumping concrete must be set up, supported, and moved with care to avoid coating damage and prevent reinforcing bars from being displaced from their appropriate position.

There will often be uncoated and epoxy-coated bars on the same construction project. The placing drawings should show the locations of the uncoated and coated bars, and projecting bars.

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9.5.5.4 Bar Supports for Epoxy-Coated Reinforcing Bars

Non-corrosive (compatible) bar supports and tie wires are usually used for epoxy-coated reinforcing bars. The purpose of the compatible types of bar supports is to minimize damage to the coating on the reinforcing bars during field placing of the coated bars, and not to introduce a potential source of corrosion at, and in close proximity to, the point of contact of the bar supports with the coated bars. The requirements for these materials should be included in the project specifications. Reinforcing bars used as support bars are usually epoxy-coated and additional details are provided in the relevant ASTM specifications. CRSI recommendations for bar supports for supporting epoxy-coated reinforcing bars are given in the next section. Coated tie wire is usually required for tying epoxy-coated bars. The coating material on the wire is usually PVC. In this regard, CRSI recommends:

a. Wire bar supports should be coated entirely with dielectric material such as epoxy or plastic, compatible with concrete.
b. Bar supports should be made of a dielectric material such as plastic or precast concrete; if precast concrete blocks with embedded tie wires or precast concrete doweled blocks are used, the wires or dowels should be coated with epoxy or plastic.

Reinforcing bars used as support bars are required to be coated with epoxy. Where indicated by the engineer, spreader bars in walls reinforced with epoxy-coated bars should also be epoxy-coated. Utilized in walls with epoxy-coated reinforcing bars, combination bar clips and spreaders should be made of corrosion-resistant material or covered with a dielectric substance (CRSI 2011).

9.5.6 Design Considerations

Typically, designers of structures utilizing epoxy-coated rebar adhere to the conventional methods specified by ACI and AASHTO. However, plant and site employees should be aware that laps and splices for epoxy-reinforcing steel require longer development and lap lengths due to the differences in bonding between a standard black bar and epoxy-coated bar.

9.5.7 Usage of Epoxy-Coated Rebar

Epoxy-coated rebar can be utilized in any structural concrete where there is an above-average possibility of rebar corrosion due to ingress of moisture, chloride, or other harmful agents. Typical usages are:

- Concrete pavements and bridge decks where deicing salts are used to keep them free from ice in cold weather.
- Marine structures like wharf, piers, jetties, dry docks, and slipways.
- Concrete structures at or near the sea.

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- Water reservoirs and effluent treatment plants (ETP).
- Drainage systems.
- Chemical plants, etc.

Photographs of a few projects where epoxy-coated rebar has been used are shown in Figures 9.4, 9.5 and 9.6.

FIGURE 9.4 Application of ECR as pile reinforcement (Photo courtesy – BSRM).

9.5.8 Issues with Epoxy-Coated Rebar

Epoxy-coated reinforcing steel was first used in Pennsylvania in 1973. ACI 318-19 and CRSI (2013) allow epoxy coating of rebar, and fusion-bonded epoxy-coating technology has been the primary defense used by most U.S. state highway agencies. It is widely used in the United States, Canada, Japan, China, India, and the Middle East. Changes to the coating and application processes that are not visible to the customer have occurred over the last three decades. These modifications have resulted in epoxy-coated reinforcing steel products that are more flexible, robust, and durable,

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FIGURE 9.5 Application of ECR in a spread footing (Photo courtesy – BSRM).

FIGURE 9.6 Application of ECR in floor slab (Photo courtesy – BSRM).

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as specified by ASTM A775 and ASTM A934 for reinforcing bar and ASTM A884 for welded wire fabric.

The Federal Highway Administration (FHWA) of the United States conducted a study (Smith and Virmani, 1996) and concluded that ECR effectively protects bridge decks from corrosion for three to twenty years of service. None of the evaluated decks exhibited any significant corrosion problem. The first bridge decks with ECR rebar exhibited no indications of distress. A study by Samples and Ramirez (1999) concluded that if an increased coating thickness of 305 to 457 micron is maintained, the damage to the coating will be minimized, which will lead to effective corrosion resistance without significantly affecting the bond behavior. An FHWA study conducted by Sohanghpurwala and Scannall (1999) on a number of bridges in Pennsylvania and New York found that the condition of epoxy-coated rebars at that time in bridge decks was found to be very good from a corrosion point of view. A seven-year-long laboratory study by FHWA on simulated bridge decks (Lee and Krauss 2004) found that epoxy-coated rebar can be used in bridge decks to achieve a 50-year maintenance-free life if both the top and bottom mesh of rebar of the deck are epoxy coated. A study on the marine substructures of four bridges was carried out by Cui *et al.* (2007), which represented a service life of 21 years. The study found no corrosion or other problems with the ECR used in these structures. The Minnesota Department of Transportation conducted a study on four bridges that were constructed between 1973 and 1978 (Pincheira *et al.* 2008). These bridges are regularly subject to deicing slats in winter. The study finds that even after a service life of about 30 years, the overall condition of the epoxy-coated bars is good to very good, with no or modest levels of corrosion activity. In only one bridge does corrosion activity appear to be moderate to severe. The majority of corroded bars were found near joints or at crack locations.

While epoxy-coated rebar has some obvious advantages in preventing corrosion of rebar, it does come with some issues as well. Epoxy coating can prevent corrosion of rebar only if it remains fully undamaged during construction. Such a *perfect* scenario is able to be simulated in laboratory investigations. In real life application of epoxy-coated rebar, the scenario is far from perfect. Firstly, it is nearly impossible to manufacture epoxy-coated rebar completely free of any defect. Even ASTM A775 specification allows one *holiday* per linear foot to be acceptable. Though there are specific guidelines for preventing damages to the coating during transporting, handling, placing, and concreting, these bars invariably incur various damages to coatings. This is because it is practically impossible for the workers to maintain all the precautions during the handling and use of epoxy-coated bars. Recognizing this fact, there are provisions for repairing the damaged coating onsite after the rebar is laid. The effectiveness of such a repair depends on how meticulously the relevant repair personnel is doing the job. It is practically impossible to detect and repair all the damages in most cases. Considering the scenario depicted in Figure 9.5, there is a good possibility that the coating is scraped off and the steel gets exposed when concrete aggregate particles are ground into the bars by the boots of workers walking over the partly cast areas. The coating may also get damaged by the shovels used by workers.

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There are numerous other such factors which prevent the coating from remaining in perfect condition. It is possible to initiate corrosion from even a small damage in the coating in a favorable environment, since the coating has no cathodic protection. Corrosion cells are set up at the damaged locations which lead to the delamination of the coating and the development of rust. The nearly unavoidable fine cracking occurring during bending, the pinholes occurring in the coating, and the inferior protective ability of the patch repaired zones and cut ends have led epoxy-coated reinforcement to be a less attractive solution (Rostam 2005). It also has the side effect that this technology will prevent the possible future use of cathodic protection, leaving no alternative but replacement of damaged members if corrosion develops.

After the invention of epoxy coating of rebar for prevention of corrosion in the early 1970s, its use became widespread around the world. A belief developed that epoxy coating shall enable a 100-year lifespan of concrete structures without major repair or maintenance. However, a few experiences that surfaced in the 1990s led engineers to think otherwise. It was reported that many bridges experienced severe corrosion problems due to poor handling of FBEC rebars in Florida (McDonald 2009). Similar poor site practices are common in many countries, especially in the developing world. Studies conducted by the National Cooperative Highway Research Program (NCHRP), for the Florida Department of Transportation document unsatisfactory performances of epoxy-coated reinforcement in bridge structures located in coastal environments (Clear *et al.* 1995). The report concluded that ECR technology cannot be relied on to provide long-term (50-plus years) corrosion protection to concrete structures. Studies of Kessler *et al.* (1993) and Weyers *et al.* (1998) had similar conclusions. A state research project in Oregon, United States (Griffith and Laylor 1999) concluded that the use of epoxy-coated rebar for coastal structures should be discontinued. A summary research report published by the Virginia Transportation Research Council (Weyers *et al.* 2006) commented that while short-term (less than two years) laboratory researches show good corrosion behavior of ECR, long-term (seven years or more) laboratory research showed that significant corrosion occurred in chloride-contaminated environments. The report also concluded that ECR will not provide long-term corrosion protection in northern American and Canadian deicing salt environments. A report by the American Segmental Bridge Association (Rostam 2005) commented that because of the lack of reliable corrosion protection performances of ECR on a long-term basis, this technology did not gain a good foothold in Europe.

The technology of coating steel rebar with epoxy to prevent corrosion emerged in the 1970s to address the need to protect concrete bridges and marine structures from rebar corrosion induced damage and to increase the durability. Experiences of using ECR to prevent rebar corrosion appear to be mixed. There are examples of both failures and successes. It appears that monitoring of structures where ECR is used as well as further research to improve the performance of ECR technology, as well as construction practices, need to be continued. Meanwhile, it is advisable that engineers and project owners should make themselves aware of the properties, pros, and cons of ECR technology before making decisions.

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9.6 ZINC-COATED (GALVANIZED) REINFORCING BARS

An important line of defense against rebar corrosion is protecting the rebar itself by hot-dip galvanizing (HDG). Hot-dip galvanized coatings create an impervious metallic zinc and zinc alloy barrier around the steel and isolate the steel surface from the surrounding concrete. It is a sacrificial coating system that corrodes itself and protects the steel from corrosion.

The earliest reported use of zinc-coated steel in concrete dates back to 1908. In the 1930s, its regular use as a reinforcing material started in the United States (Galvanize It 2022). One early example was the construction of concrete water tanks where galvanized wire was used to pre-stress the tank wall. In the decades following World War II, the use of galvanized rebar increased, and by the 1960s and early 1970s, a substantial amount of steel reinforcement was being galvanized for use in bridge and highway construction in the snow-belt states of the United States and Canada. In the late 1970s, when the FHWA classified galvanizing as an experimental system, its use decreased somewhat. This ruling was withdrawn in 1983, allowing states greater freedom in the selection of corrosion protection systems, including galvanization.

Since then, and especially over the last 25–30 years, galvanized reinforcement has been consistently used in a wide variety of concrete construction types and exposure conditions across the globe. Galvanized reinforcement has been widely accepted in recent years, as evidenced by the worldwide publication of a number of international standards for the use of zinc-coated (i.e. galvanized) reinforcement, as well as the existence of numerous Codes and Specifications pertaining to galvanized reinforcement that have been published by federal and state bodies, particularly in North America.

Galvanized rebar is a *sacrificial* coating system that has numerous advantages over traditional uncoated rebar including:

a. Zinc has a higher chloride concentration threshold than uncoated steel for corrosion. This significantly delays the onset of corrosion caused by chloride penetration to the galvanized surface of the rebar.
b. The corrosion rate of zinc in concrete is lower than that of black steel, and zinc corrosion products do not produce the same deteriorating internal stresses as steel corrosion products do in concrete.
c. In the event of a breach in the zinc coating, the surrounding zinc will corrode sacrificially and electrochemically shield the exposed steel from corrosion. As a result, galvanized coatings are impervious to iron corrosion products, unlike with other barrier coatings (e.g., epoxy).
d. The superior corrosion resistance of galvanized rebar permits greater tolerance in concrete placement and variability.
e. The zinc coating protects the rebar from corrosion before it is embedded in concrete.

These properties of galvanized rebar significantly reduce the risk of rebar corrosion, which causes rust stains, concrete cracking, and spalling. Zinc coating of rebar

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extends the maintenance-free life of a concrete structure and significantly reduces its total cost of ownership over its lifetime.

9.6.1 Hot-Dip Galvanizing (HDG)

Several methods exist for applying zinc coating to steel, including hot-dipping, electroplating, spraying, and mechanical alloying. Each method generates a unique type of coating that differs in structure, thickness, and performance, as well as its expected lifespan under different exposure conditions. This is because the thickness and metallurgical composition of a galvanized coating largely determine its durability.

Hot-dip galvanizing is the most common method and should be preferred for coating structural steels and reinforcing bars. The coating produced by hot-dipping, which is metallurgically bonded to the steel and typically around 100 microns thick, adheres strongly to the base steel and is quite tough and damage-resistant. It is important to remember that galvanizing is commonly used to refer to the zinc coating of steel. As it does not specify the coating method when used alone, it can be interpreted as permitting coating by any of the available methods such as electroplating or continuous galvanizing. In order to achieve the desired coating thickness and morphology, it is necessary to be precise when specifying galvanizing. Consequently, *hot-dip galvanizing* should be the preferred method for reinforcing steel, as well as the majority of structural steel sections.

In hot-dip galvanizing method, the cleaned bar of steel is submerged in a bath of molten zinc maintained at a temperature of about 450°C (842°F). A sequence of iron-zinc alloy layers are produced as a result of the metallurgical reaction that takes place between the steel bar and the zinc. As shown in Figure 9.7, there are four typical layers present from the steel substrate outwards, including the γ-layer (75% Zn, 25% Fe), the δ-layer (90% Zn, 10% Fe), the ζ-layer (94% Zn, 6% Fe), and the η-layer

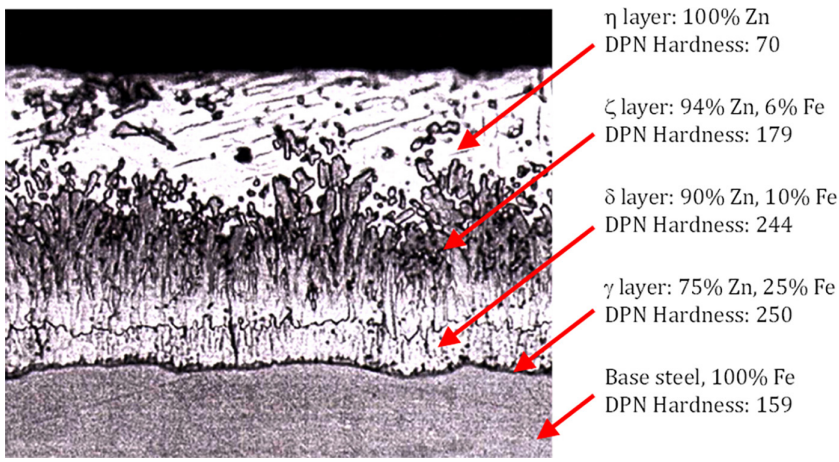
η layer: 100% Zn
DPN Hardness: 70
ζ layer: 94% Zn, 6% Fe
DPN Hardness: 179
δ layer: 90% Zn, 10% Fe
DPN Hardness: 244
γ layer: 75% Zn, 25% Fe
DPN Hardness: 250
Base steel, 100% Fe
DPN Hardness: 159

FIGURE 9.7 Photomicrograph of zinc coating (adapted from Intl. Zinc Assoc. 2022).

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(100% Zn). Each of these layers is composed of a different percentage of Zn relative to Fe (Tang 2016).

The distinctive structure of HDG coating results in a multitude of significant benefits. To begin, it forms metallurgical links to the steel beneath it, which prevents under-film corrosion from occurring. Second, the iron-zinc alloy layers are tougher than the steel that lies beneath them, which results in the production of a coating that is both tough and resistant to abrasion. In this respect, the processes of shipping and handling do not demand any additional precaution.

9.6.2 How Galvanizing Protects Rebar from Corrosion

Hot-dip galvanization is used to protect steel because of the coating's dual purpose action. Galvanizing creates a durable, metallurgically-bonded zinc coating that completely covers the steel surface and protects it from corrosive effects in the environment. Additionally, even in cases of minor coating damage or discontinuity, steel is still protected by the cathodic action of zinc. It should be noted that hot-dip galvanized reinforcing steel performs very differently in concrete than it does in atmospheric conditions.

9.6.2.1 Barrier Protection

Zinc is distinguished by its amphoteric nature and capacity for passivation as a result of the development of barrier reaction product films. Zinc's reaction with new cement causes passivity by creating a layer of zinc corrosion products that act as a diffusion barrier.

9.6.2.2 Sacrificial Cathodic Protection

The galvanized coating offers cathodic protection to exposed steel because zinc is anodic to steel. In the presence of an electrolyte, when zinc and steel are connected, the zinc slowly corrodes while the steel is kept safe. When small areas of steel are exposed due to cuts, drill holes, scratches, or extreme surface abrasion, zinc's sacrificial action provides protection. Until all of the nearby zinc is consumed, the steel is cathodically protected from corrosion.

Normally, steel and chromated zinc behave passively in concrete's extremely alkaline environment. However, the penetration of chloride ions to the metal surface can override this passivity and start sacrificial corrosion of zinc or the rusting of steel. The main justification for using galvanized steel reinforcement is the vulnerability of concrete structures to chloride intrusion.

Galvanized steel can withstand exposure to chloride ion concentrations that are at least four to five times higher (on average) than the level of chloride that causes corrosion in reinforcement made of ordinary black steel. Galvanized reinforcement can stay passivated at a lower pH, providing significant protection against the effects of concrete carbonation, whereas black steel in concrete typically depassivates below a pH of 11.5.

It is generally acknowledged that the combination of chloride tolerance and carbonation resistance accounts for the superior performance of galvanized reinforcement

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over black steel reinforcement. The total lifespan of a galvanized coating in concrete consists of the time it takes for the zinc to depassivate (which is longer than it would be for black steel due to its higher tolerance for chloride ions and carbonation resistance) as well as the time it takes for the zinc coating to corrode as it serves as a sacrifice to protect the underlying steel. Localized corrosion of the steel won't start until the coating has been completely consumed off in that area of the bar.

9.6.2.3 Chemistry of Zinc Coating on Rebar in Concrete

Galvanized rebar in concrete provides corrosion protection due to a number of advantageous effects. The fact that zinc coatings have a significantly higher chloride threshold (2–4 times) than uncoated steel before they begin to corrode is crucial. Galvanized rebar is also immune to the pH-lowering effects of carbonation as concrete ages because zinc has a much wider pH passivation range than steel. Even if the zinc coating does begin to corrode, it does so at a much slower rate than uncoated steel.

Less expansive than iron corrosion products, zinc's corrosion byproducts are loose, powdery minerals that can migrate from the surface of galvanized rebar into the surrounding concrete matrix. As a result, very little physical damage to the nearby concrete results from the zinc coating corroding. Additionally, there is data to support the idea that the diffusion of zinc's corrosion products helps fill pore spaces at the concrete/rebar interface, making this region less permeable and reducing the transport of aggressive species like chlorides through this interface zone to the zinc coating. Galvanized rebar's excellent bond strength with concrete is also explained by reactions between zinc and concrete and the diffusion of corrosion products that results from these reactions.

A compound called calcium hydroxyzincate is produced when zinc reacts with wet concrete, along with the evolution of hydrogen. This anti-corrosion product is insoluble and safeguards the zinc underneath (provided that the pH value is below 13.3 in the surrounding concrete). Research has shown that some of the coating's pure zinc layer dissolves during this initial reaction phase before coating passivation and concrete hardening take place. However, once the concrete hardens and the hydroxyzincate coating has formed, the initial reaction stops. Studies on galvanized rebar recovered from field structures show that the coating maintains this passive state even when exposed to high chloride levels in the nearby concrete for long periods of time (Tang 2016).

The zinc surface can be passivated using a variety of proprietary post-treatments for concretes with high pH levels or where some background chlorides are expected as a defense against excessive hydrogen evolution that could, in extreme cases, weaken the pullout strength of the bar. Research has found no statistically significant difference between galvanized rebar that was passivated and that was not passivated in bond strength for typical concrete conditions (Kayali and Yeomans 2000).

In addition to reducing the HDG coating thickness, chemical reactions in fresh concrete may shorten the service life of structures. In order to passivate the zinc surface, a specific treatment is specified in ASTM A767. Typically, freshly galvanized steel is quenched in sodium dichromate-containing water. The process is called

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chromating. However, the usage of chromates is very restricted due to rising concerns about the associated risks to human health and environmental problems.

According to both laboratory and field tests, the critical chloride corrosion threshold of HDG coated steel bar is greater than that of conventional steel bar, and the average time to corrosion initiation in bridge decks for HDG coated steel is 4.8 years, compared to 2.3 years for bare steel bar. The ASTM A767/A767M *Standard Specification for Zinc-Coated (Galvanized) Steel Bars for Concrete Reinforcement* mandates the usage of HDG coated steel reinforcing bars in concrete constructions.

9.6.3 Physical Properties of HDG Coatings

9.6.3.1 Metallurgical Bond

Because of its unique metallurgical bond with the steel, hot-dip galvanizing is a factory-applied coating that offers a combination of qualities unmatched by other coating systems. A typical section of hot-dip galvanized coating is depicted in the photomicrograph in Figure 9.7. A series of zinc-iron alloy layers that are mechanically bonded to the base steel at a pressure of about 3600 psi make up the galvanized coating. Since the galvanizing process creates a metallurgical bond, under-film corrosion is prevented. Epoxy coatings, on the other hand, merely apply a film on top of the steel. As shown in Figure 9.8, corrosion starts as soon as the epoxy film is damaged and the bare steel is exposed. The uncovered steel corrodes, creating a pocket of rust and a blister as the coating is lifted from the metal surface. This blister will enlarge until the coating has completely failed. When a galvanized surface is damaged, the zinc coating gives itself up to protect the steel; this behavior lasts as long as there is zinc in the vicinity of the damaged area.

9.6.3.2 Impact and Abrasion Resistance

The ductile outer zinc layer provides good impact resistance to the bonded galvanized coating. The micrograph mentioned in Figure 9.7 shows the typical hardness values of a hot-dip galvanized coating. Zeta, gamma, and delta layers' hardnesses are actually higher than that of the base steel, providing exceptional resistance to coating abrasion damage.

9.6.3.3 Corner and Edge Protection

On non-galvanized products, corrosion frequently starts at the corners or edges. No matter the method of application, epoxy coatings are thinnest at the corners and edges. At these locations, the galvanized coating will be at least as thick, if not thicker, than on the overall surface. This offers these critical locations the same protection – if not more than before.

9.6.3.4 Complete Coating

Because galvanizing is performed by total immersion, all surfaces of the item are fully coated and protected, including those that are inaccessible and difficult to reach

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with brush-on or spray-applied coatings. In addition, the integrity of any galvanized coating is guaranteed because zinc will not metallurgically bond to dirty steel. As the work is withdrawn from the molten zinc, any uncoated areas become immediately apparent. When necessary, adjustments are made on the spot in order to deliver a fully protected item to the job site.

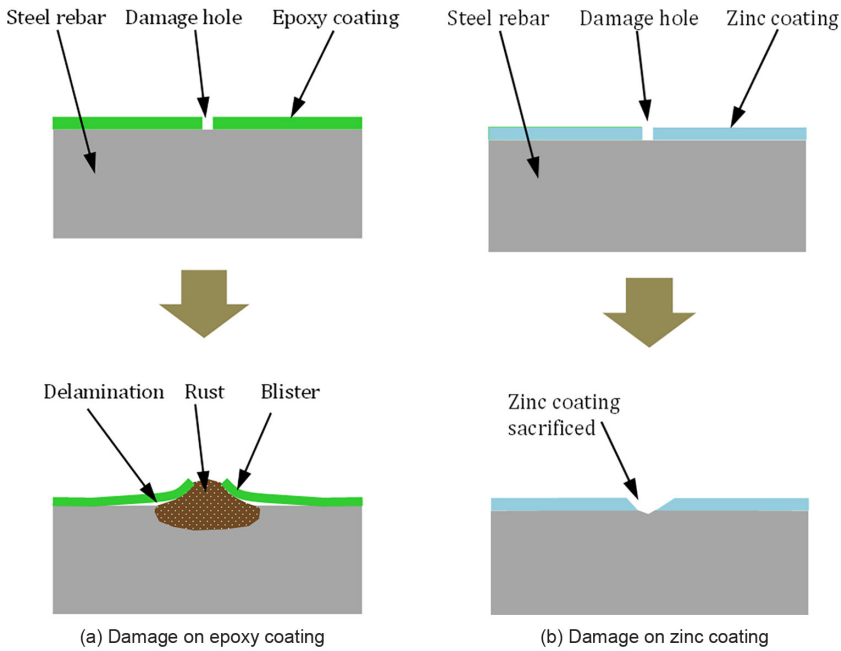

FIGURE 9.8 Comparison of the behavior of damaged epoxy and zinc coating.

9.6.4 Mechanical Properties of HDG Rebar

9.6.4.1 Ductility and Yield/Tensile Strength

The ductility and strength of reinforcing steel are crucial for preventing the failure of reinforced concrete due to brittleness. Hot-dip galvanizing has been shown to have little effect on the mechanical properties of steel reinforcing bars, including tensile, yield, and ultimate strength, as well as ultimate elongation and bend requirements, provided that steel selection, fabrication practices, and galvanizing procedures are properly carried out following standard specifications.

9.6.4.2 Fatigue Strength

Extensive experimental research on the fatigue resistance of galvanized steel reinforcement reveals that deformed reinforcing steel exposed to an aggressive environment prior to cyclic tension testing performs better when galvanized (Sarja *et al.* 1984).

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9.6.4.3 Bond Strength

Reinforced concrete structures cannot function properly without a strong bond between the steel reinforcing the structure and the concrete itself. Protective coatings on steel must not weaken the bond between the steel and coatings. Concrete made with Portland cement and galvanized, or black steel rods, has been studied for evaluating bonding characteristics by many researchers (Kayali and Yeomans 2000). The results of these studies indicate:

- Depending on the zincate/cement reaction, the time needed to develop a full bond strength between steel and concrete may occasionally be longer for galvanized bars than for black bars.
- Both galvanized and black deformed bars have the same fully developed bond strength.

9.6.5 Application of HDG Rebar

In a variety of reinforced concrete elements and structures, galvanized rebar and other fittings (such as bolts, ties, anchors, dowel bars, and piping) have been widely used. There are specific situations where galvanizing reinforcement is likely to be a wise engineering choice, such as:

- Transport infrastructure, including bridge decks and pavements, road surfaces, crash barriers, and tunnel linings.
- Architectural building features and lightweight precast cladding components.
- Prefabricated buildings with tilt-up construction and modules for kitchens and bathrooms.
- Concrete elements that are submerged or buried and are affected by groundwater and tides.
- Coastal and marine structures like docks, jetties, offshore platforms, marinas, floating pontoons, and moorings, as well as sea walls and coastal balustrades.
- Structures with a high risk of failure in hostile environments, such as chemical plants, paper mills, and water and sewage treatment facilities.
- Various buildings, including water storage tanks, facilities, coal storage bunkers, cooling towers, and chimneys.

There are numerous instances of galvanized rebar being used successfully in a range of reinforced concrete buildings, structures, and general construction projects throughout the world, some of which are shown in Figures 9.9 and 9.10.

9.6.6 Using HDG Rebar in Concrete Structures

9.6.6.1 Design and Construction

The design specifications and installation techniques used for structures using galvanized steel reinforcement should be no less stringent than those used for

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FIGURE 9.9 Galvanized rebar in pavement (Photo ©Trissi1234, Wikimedia Commons CC by SA 4.0, available at https://commons.wikimedia.org/wiki/File:Br%C3%BCckenkappe_mit_feuerverzinkter_Bewehrung.jpg, accessed 16 Sept. 2024).

FIGURE 9.10 Galvanized rebar in the parapet of box girder bridge.

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structures using uncoated steel reinforcement (see the AGA publication, *Suggested Specification for Hot-Dip Galvanizing Reinforcing Steel*). Additionally, when using galvanized steel, there are some unique guidelines that must be followed. For the benefit of designers, engineers, contractors, and inspectors, the following advice is provided. They only address the unique considerations that result from the use of galvanized steel and are meant to be a supplement to other codes and standards dealing with the design, fabrication, and installation of reinforced concrete structures.

Construction activities should be carried out on the job site in a way that minimizes coating damage. The project specifications should contain any special requirements for galvanized bars, similar to or parallel to the special requirements for epoxy-coated bars, which were discussed previously. These special requirements for galvanized bars should cover compatible bar supports and tie wire, splicing and mechanical splices, field bending and straightening, limits on the amount of repaired damaged coating, and repair of damaged coating.

9.6.6.2 Steel Selection

One or more of the following standard ASTM specifications must be met by the concrete reinforcing steel before being galvanized:

- ASTM A615: *Standard Specification for Deformed and Palin Billet-Steel Bars for Concrete Reinforcement.*
- ASTM A616: *Standard Specification for Rail-Steel Deformed and Plain Bars for Concrete Reinforcement.*
- ASTM A617: *Standard Specification for Axle-Steel Deformed and Plain Bars for Concrete Reinforcement.*
- ASTM A706: *Standard Specification for Low-Alloy Steel Deformed and Plain Bars for Concrete Reinforcement.*

9.6.6.3 Detailing

Galvanized reinforcing steel detailing should adhere to the design specifications for uncoated steel bars and to customary industry practice in accordance with the Concrete Reinforcing Steel Institute's recommendations (CRSI 2015). Due to the equal bond strength to concrete, hot-dip galvanized reinforcing steel's overlap lengths are identical to uncoated steel reinforcement's overlap lengths.

9.6.6.4 Bending of Bars

Bends or hooks ought to be rounded, not pointed. CRSI's recommendations should be followed when cold bending. Bars that are to be cold bent before being galvanized must be made to a bend diameter not smaller than those listed in Table 9.1. Before hot-dip galvanizing, if the material is stress-relieved at a temperature between 482°–566°C for one hour per 25 mm of bar diameter, it can be cold bent more tightly than shown.

Galvanizing before bending may result in some flaking and cracking of the galvanized coating at the bend. The integrity of the coating may also be impacted by how quickly the object is bent. Slower bend speeds are best for maintaining the

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TABLE 9.1
Minimum suggested bend diameters before galvanizing (ASTM A767)

Bar No.	Grade 60, 75 and 80 [420, 520, 550]
3, 4, 5, 6	$6d_b$
7, 8	$8d_b$
9, 10	$8d_b$
11	$8d_b$
14, 18	$10d_b$

galvanized coating. A little bit of galvanized coating flaking and cracking in the bend area is not a reason for rejection, according to ASTM A767, *Specification for Zinc-Coated (Galvanized) Steel Bars for Concrete Reinforcement*. As stated in ASTM A780, *Practice for Repair of Damaged and Uncoated Areas of Hot-Dip Galvanized Coatings*, any flaking or cracking can be fixed. For Class 2 coating, the zinc coating shall not peel or flake off in any bend test meeting the requirements of ASTM Specifications A615, A706, or A996.

9.6.6.5 Dissimilar Metals in Concrete

When using galvanized reinforcement in concrete, one must also take into consideration the possibility of producing a bimetallic couple between zinc and bare steel (i.e., at a break in the zinc coating or direct contact between galvanized steel and black steel bars), as well as between zinc and other dissimilar metals. This is an important consideration because it can occur when the zinc coating on the steel is broken. As long as the two metals are passivated, a bimetallic couple of this kind in concrete shouldn't be expected to show corrosive reactions. The concrete depth to the zinc/steel contact should not be less than the cover needed to protect black steel alone under the same conditions. Galvanized reinforcement should not, therefore, be coupled directly to copper, black steel reinforcement, or any other dissimilar metal when used in concrete. Galvanized hardware (e.g. bar supports) for bars should be used. Tie wire needs to be annealed wire, preferably galvanized, and 16 gauge or larger. Tapes made of polyethylene and other comparable materials can be used, if preferred, to insulate different metals.

9.6.6.6 Storage

It is possible to store galvanized bars outside without causing a reduction in their resistance to corrosion. Because of the general ease with which they can be stored, it is possible to store standard lengths in order to have them readily available when required. Because it is so resistant to abrasion, galvanized reinforcing steel can be handled and installed in exactly the same way as black steel reinforcement can. This is yet another important advantage of galvanized reinforcing steel.

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9.6.6.7 Installation Welding

The Structural Welding Code – Reinforcing Steel (AWS D1.4) contains requirements and procedures for welding galvanized bars. If proper precautions are taken, welding galvanized reinforcement poses no difficulty. Steps include welding at a slower rate and maintaining adequate ventilation (the ventilation normally required for welding operations is considered adequate). More details are outlined the publication title *Welding and Hot-Dip Galvanizing* by the American Galvanizing Association.

9.6.6.8 Local Repair of Coating

Local removal of the galvanized coating in the vicinity of welds, bends, or sheared ends will not have a significant impact on the protection offered by galvanizing, provided the exposed surface area is small in comparison to the adjacent galvanized steel surface area. The area can be repaired in accordance with ASTM A780 when the exposed area is excessive, and there are visible gaps in the galvanized coating.

9.6.6.9 Removal of Forms

Due to the fact that cement with naturally low levels of chromates may react with zinc and retard hardening and initial set, it is essential that forms and supports are not removed before the concrete has developed the necessary strength to support itself. Normal form removal practices may be used if the cement contains at least 100 ppm of chromates in the final concrete mixture or if the hot-dip galvanized bars have been chromate passivated in accordance with ASTM A767, Section 4.3.

9.6.7 Thickness of Zinc Coating

The thickness of the galvanized coating is the primary determinant of the product's service life. The greater the thickness of the coating, the longer it will resist corrosion. The relationship for steel embedded in concrete is approximately linear. Table 9.2 summarizes the minimum coating mass or weight requirements for reinforcing bars specified in ASTM A767. Notably, zinc coating increases the unit mass (mass/length) of rebar by approximately 6 to 10%.

There are various ways to determine the thickness of the galvanized coating. The testing approach will probably be determined by the size, shape, and quantity of the parts to be tested. The most popular technique for determining the coating thickness

TABLE 9.2
Minimum zinc coating thickness for rebar (ASTM A767)

Usage/ Type	Bar size	Minimum mass of zinc coating, gm/m²	Minimum coating thickness, microns
Class 1	Bar #3 [10]	920	129
	Bar #4 [13] and larger	1070	150
Class 2	Bar #3 [10] and larger	620	86

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on reinforcing steel bar and assembly is the ASTM-compliant magnetic thickness gauge. Details are described in ASTM E376, *Standard Practice for Measuring Coating Thickness by Magnetic-Field or Eddy-Current (Electromagnetic) Examination Methods.*

9.6.7.1 Factors Affecting Zinc Coating Thickness

The zinc coating thickness is affected by a number of factors, some of which the galvanizer can control and some of which it cannot. The temperature of the zinc bath and the rate at which the reinforcing bar is removed from the zinc bath are the two that the galvanizer can easily control. The coating thickness is somewhat influenced by the surface's roughness. As a result, parts with rough surfaces from excessive pickling may form thicker zinc-iron layers.

The chemical composition of the steel, specifically its silicon and phosphorous content, is one factor that the galvanizer has no control over but which has a significant impact on the final results of the galvanized coating. The zinc-iron alloy layers tend to grow more quickly at certain levels of silicon and phosphorous, so the coating keeps getting thicker the longer the reinforcing steel is submerged in the zinc bath. According to ASTM A385 *Practice for Providing High-Quality Zinc Coatings*, there are two ranges of silicon content that are recommended for steels to be galvanized: below 0.04% or between 0.15% and 0.22% (hot-dip). Steels with silicon contents outside of these ranges typically result in thick coatings with matte gray surfaces, which represent an intermetallic zinc-iron coating with little to no free zinc outer layer.

Hot-dip galvanized coatings that are more than 12 mils (305 microns) thick are susceptible to damage from rough handling even though service life is linear to the thickness of the zinc coating. Where there is external stress placed on the bars, these thick coatings may experience flaking of the outer zinc-free layer. In more severe circumstances, only a small portion of the original coating thickness may remain after the coating fractures at the boundary between distinct zinc-iron intermetallic layers.

Combining different bar sizes in a reinforcing steel assembly is another factor that affects coating thickness in hot-dip galvanization method. Zinc coating will form on bars of various sizes at varying rates. Smaller bars in the assembly typically have thicker than normal coatings, which will be more obvious if the silicon levels are outside the recommended ranges. Thicker sections of the assembly must be kept in the zinc bath for a longer period of time in order to develop the minimum required thickness.

9.6.7.2 Coating Appearance

According to ASTM A123 and A767, the zinc coating must be flawless, free of blisters, flux spots, and notable dross inclusions. For maximum corrosion protection, the galvanized coating must be continuous. Wire racks, chain slings, or other holding devices must be used in handling techniques for galvanizing in order to lower material into the zinc bath. The galvanized item may acquire marks from chains, wires, and specialized fixtures used to handle pieces. Unless they have exposed the bare steel or created a handling hazard for erection personnel, these marks are not

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necessarily harmful to the coating and are not a reason for rejection. These areas can, if necessary, be repaired in accordance with ASTM A780.

A matte gray finish is not a reason for rejection, according to the same specifications. Steel with a high silicon content will have a matte gray finish and occasionally a rougher surface, which indicates an accelerated growth of the zinc-iron intermetallics. Additionally, cold-working could produce a matte gray appearance. The main criterion for judging a galvanized coating's overall appearance and determining its suitability should be its capacity to fulfill its primary goal of offering corrosion protection. Additionally, the performance of the galvanized coating or the reinforcing steel embedded in concrete is unaffected by the matte finish or rougher surface.

9.6.8 Testing of Galvanized Rebar

Different properties and features of hot-dip galvanized rebar may be tested in addition to visual inspection. To make sure that specific lots of steel comply with applicable standards or to test lots before they are put to use, these tests may need to be carried out. These tests should be carried out by accredited labs with knowledge of the specific test protocols.

9.6.8.1 Bond Strength Test

According to ASTM A944, *Test Methods for Comparing Bond Strength of Steel Reinforcing Bars to Concrete Using Beam-End Specimens*, the bond between the hot-dip galvanized reinforcing bar, and the concrete can be evaluated. The deformation of the bar and not the actual bond between the zinc and concrete is what determines how strong the bond is. The bond between the zinc and concrete becomes crucial for plain bars without deformation. Numerous tests have been conducted on the pullout strength of hot-dip galvanized reinforcing steel, and the results show that the bond strength is on par with or superior to that of black steel.

9.6.8.2 Chromate Finish Test

If chromate coating is necessary, the method described in ASTM B201, *Practice for Testing Chromate Coatings on Zinc and Cadmium Surfaces*, may be used to confirm the presence of chromates. Bars that have been galvanized and then chromated may not show any chromate by the time they are embedded in concrete because chromate conversion coatings deteriorate fairly quickly. As the chromate weathers away, zinc carbonate will start to form.

9.6.8.3 Embrittlement Tests

Stronger bars that have undergone extensive cold working could become embrittled during the galvanizing process. ASTM A143 *Safeguarding Against Embrittlement of Hot-Dip Galvanized Structural Steel Products and Procedure for Detecting Embrittlement* and ASTM A767 both contain instructions for fabricating bars that will be hot-dip galvanized. When embrittlement is suspected, ASTM A143 specifies the proper test procedure to find out if it is present.

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9.6.8.4 Accelerated Aging Test

In numerous zinc-coated steel applications, efforts have been made to develop the proper test method to establish an appropriate accelerated lifetime. ASTM B117 Practice for Operating Salt Spray (Fog) Apparatus, is one of the test procedures for corrosion prevention systems. The salt spray standards G85-02, *Standard Practice for Modified Salt Spray (Fog) Testing* and B117 are under the purview of ASTM Committee G-1 on Corrosion of Metals. The following resolution was approved by the committee regarding the application of B 117:

> *Confirms that results of salt spray (fog) tests, run according to ASTM standard designation B 117, seldom correlate with performance in natural environments. Therefore, the committee recommends that the test not be used or referenced in other standards for that purpose.*

Zinc-coated steel cannot be tested accurately with salt spray because it accelerates the incorrect failure mechanisms. The zinc coating cannot form patina layers without a proper wet/dry cycle; the absence of a patina layer allows continuous attack of the zinc metal and gives a very low prediction of the lifetime of the zinc coating. Correlation to salt spray test results is also impossible because the corrosion rate of zinc embedded in concrete is much lower than that of zinc in atmospheric conditions and has never been subject to long-term analysis.

9.6.9 Remarks on Using HDG Reinforcement in Concrete Structures

Galvanizing is one of a number of methods that are employed to prolong the life of reinforced concrete structures subjected to aggressive environments that cause reinforcement corrosion. This could increase the time to first repair in a typical construction from 15 years to at least 44 years, and probably much longer (Broomfield 2004). Like epoxy coating, galvanization of rebar for corrosion protection of steel in concrete is not a perfect technology. And unlike epoxy, the zinc coating is sacrificial. The performance of galvanized steel reinforcing bars against corrosion depends upon the intactness and thickness of the zinc coating. Both AASHTO and ASTM have specifications for zinc coating. However, the thickness specified by these standards may not be sufficient in severe chloride environments. Hence, considering severe cases of corrosive environments, a greater minimum thickness or mass of zinc coating needs to be specified. Another issue that must be kept in mind is that the zinc coating should preferably be applied using the hot-dipping method. Other zinc coating methods such as electroplating or the continuous galvanizing technique generally produce a thinner and less robust coating which may not serve the desired purpose (AGA 2013).

9.7 DUAL COATED REBAR

Dual coating of steel reinforcement is a combination of zinc and epoxy coating. Black rebar is first applied with a thermal-spray coating of zinc immediately followed by the application of fusion-bonded epoxy coating. This technology emerged

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at the beginning of the twenty-first century with an objective to provide a stronger protective coating to rebar to fight against an enhanced corrosive environment and to ensure longer service life. The first standard specification for dual coating of rebar was issued in 2008 as ASTM A1055-08 *Standard Specification for Zinc and Epoxy Dual Coated Steel Reinforcing Bars.* Dual-coated steel reinforcing bars are available in all U.S. conventional bar sizes and metric sizes. A yellow coating is generally used to identify bars that meet ASTM A1055. Handling, transportation, storing, and structural design principles followed for dual coated rebar are similar to those of epoxy-coated bars.

The following procedure is used to manufacture dual-coated steel bars. The bars are cleaned, and the surface is heated to approximately 220°C before being sprayed with a zinc arc spray, followed by an electrostatic spray containing fine epoxy powder. The powder is drawn to the zinc layer by electrostatic forces. When the epoxy comes into contact with the heated zinc-sprayed bars, it melts and fuses, resulting in the formation of a thermosetting polymer. The resulting dual coating is significantly more uniform in thickness than other methods could achieve (CRSI 2015).

In accordance with ASTM A1055, the minimum thickness required for the zinc-alloy coating ranges from 36 to 50 microns, depending on the method of application. The total coating thickness for the zinc-alloy and epoxy-coating layers shall be 180 to 300 microns for bar sizes #3 to 5 and 175 to 400 microns for #6 to 18. This is the same thickness range required for epoxy-coated reinforcing.

Ideally, the inner zinc layer provides cathodic protection for the steel in the event of damage to the outer epoxy-coating layer. This type of alternative reinforcement is relatively new; therefore, little performance data exist for this material. An experimental study over a period of 105 weeks on various coated rebar by Clemena (2003) and Clemena and Virmani (2004) found that the corrosion resistance of dual coated rebar is superior to both ECR and HGD rebar, and the protective capability is similar to stainless steel rebar. However, it must be kept in mind that the study was done in a laboratory environment for a short period of time. Further study for a longer period of time is necessary for a better understanding of the actual corrosion resistance performance of dual-coated rebar.

9.8 STAINLESS STEEL-CLAD REBAR

As the name suggests, stainless steel-clad (SSC) rebar are carbon steel bars with stainless steel coating. It is produced in a different manner than the other types of coated reinforcement. First, a strip of stainless steel is formed into a tube with an approximate diameter of 4 inches and the seam is welded together by plasma. Carbon steel filings are then compacted into the tube with a hydraulic ram. Once filled, the tube is heated and hot-rolled. The hot rolling has two functions. Firstly, it fuses the carbon steel fillings together into a solid core, and secondly, it creates a metallurgical bond between the stainless steel skin with the carbon steel core (Clemena *et al.* 2003). Stainless steel-clad reinforcing was developed to obtain the benefits of stainless steel without its high cost. It can be less expensive than solid stainless steel, but it is a bit more expensive than epoxy-coated rebar (Lute *et al.* 2018).

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SCR can provide corrosion resistance comparable to solid stainless steel; however, the production of high-quality SCR can be challenging as corrosion resistance can be diminished if the fabrication of the clad bar is not completed properly. Occasionally, gaps may form between the cladding and core material. This can lead to cracking and voids which can decrease corrosion resistance and ultimate service life. Additionally, the manufacturing process results in stainless steel cladding along the length of the bars and an exposed carbon steel core at the ends of the bars. These ends must be properly sealed to prevent corrosion of the carbon steel core.

The results of early research conducted by Rasheeduzzafar *et al.* (1992) indicate stainless-clad reinforcement has highly superior corrosion resistance compared to traditional galvanized and epoxy-coated bars. In the study, cracking was noted in the traditional, galvanized, and epoxy-coated bars within six months, whereas no cracking was noted in the stainless steel-clad bar specimens after seven years. A later study spanning over three years, conducted by the Virginia Transportation Research Council (Clemena *et al.* 2003) concluded that the stainless steel-clad bars exhibited the same corrosion resistance as solid stainless steel and provided a chloride corrosion threshold at least 16 times that of traditional black steel.

The performance of SCR has been shown to be superior to ECR. This is due to the toughness of the cladding, which prevents most of the chipping and cracking that is often seen during the handling, placing, and concreting of epoxy-coated bars. However, problems with stainless-clad rebar result from imperfect bonding between the stainless steel skin and carbon steel core, which can make the carbon steel vulnerable to corrosion (Van Dyke *et al.* 2017). Damage to the cladding is still possible, however, and the material should be handled with care during fabrication to minimize damage (Darwin *et al.* 2013). Also, the use of plastic caps instead of two-part epoxy for sealing the ends of clad bars can result in handling related corrosion issues. Plastic end caps can often fall off during handling and placement, exposing the carbon core to potential chloride ingress.

One of the primary issues with SSC is its very limited availability. SSC rebar is produced only by a very few manufacturers in the world. Therefore, project owners must take into consideration the additional lead time required to order and procure SSC rebar.

9.9 CHEMICALLY REACTIVE ENAMEL

Over 3,000 years ago, the process of enameling was utilized to paint the surfaces of things for aesthetic purposes. During the industrial revolution, its usage as a coating material applied to iron and steel for objectives such as corrosion protection, heat resistance, abrasion resistance, hygiene, and so on, had emerged.

Enamel is employed in a variety of applications today, including household goods, industrial environments, and the construction industry. Enamel is used to coat cooking equipment in houses because it is easy to clean, inhibits bacteria growth, does not absorb odors, and is unaffected by culinary acids. It's often used in industry to protect the interiors of tanks, boilers, ovens, tubes, and stove components. Enamel is employed as a decoration for cladding structures or interior decor in the construction

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industry because it combines a rigid steel base with varied surface effects like roughness, reflectivity, salt and pepper effects, and metalized hues (Tang 2016).

Enamel is applied to steel surfaces using either a wet or dry procedure. Steel is dipped into a vat containing enamel slurry, or enamel slurry is sprayed on the surface of the steel in the wet process. The enamel slurry-coated steel is then placed in a furnace heated to between 750 and 850°C, where it dries, melts, and flows. It is taken out of the furnace after several minutes, cools, and hardens to a smooth, durable vitreous enamel on the steel. In the dry application, enamel powder is electrostatically sprayed onto the steel surface before putting it inside the furnace. The typical microstructure of enamel coating on a steel substrate is shown in Figure 9.11. Due to chemical reactions between the oxides in the enamel frits and the carbon in the steel during the burning process, some air bubbles are present.

In 2006, the use of enamel coating to prevent steel rebar from corrosion in concrete constructions was examined for the first time. As demonstrated in Figure 9.11a, commercially available enamel was changed by adding Portland cement to form a chemically reactive enamel (CRE) layer. This CRE coating improves both the corrosion resistance and the binding strength of the steel rebar with the surrounding concrete. However, adding Portland cement to enamel coatings changes the microstructure from solid with isolated air bubbles to porous with connected channels (Figure 9.11a), lowering corrosion resistance. As a result, double-layer enamel coatings (Figure 9.11b) and sand particle-modified enamel coatings (Figure 9.11c) have been proposed and studied. The double enamel is made up of an inner black enamel and an outer CRE, with the inner black enamel improving corrosion resistance and the outer CRE increasing the bond strength. The sand particle was employed to boost binding strength through the rough surface while maintaining corrosion resistance at the same time.

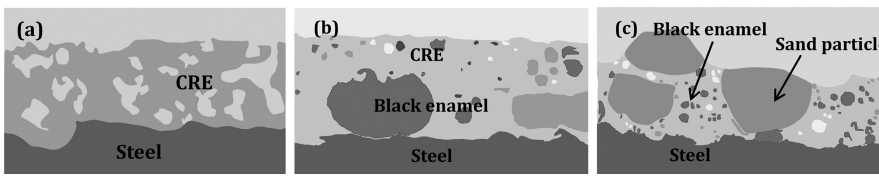

FIGURE 9.11 Microstructure of (a) chemical reactive enamel (CRE), (b) double enamel, and (c) sand particle modified enamel (adapted from Tang 2016).

Despite these efforts, enamel coating has one fatal flaw: it is fragile, which restricts its use. Bending of the rebar easily breaks the enamel and causes delamination. It is easily damaged during shipment and handling, and pitting corrosion is a possibility.

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QUESTIONS

9-1. What is the main objective of using coated reinforcing steel in concrete structures? Provide a brief history of coated reinforcement bars.

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9-2. List and discuss different types of coated bars in accordance with ASTM standards.
9-3. What are the raw materials used in fusion-bonded epoxy coating? Discuss.
9-4. Briefly describe the step-by-step process of manufacturing epoxy-coated rebar.
9-5. List the required tests for the quality control of epoxy-coated reinforcing bars according to ASTM specifications.
9-6. Explain the handling process for epoxy-coated rebar during manufacturing and shipment.
9-7. Briefly describe the procedure for handling, storing, and placing epoxy-coated rebar at site.
9-8. Explain the functions and requirements for bar supports and tie wires for epoxy-coated rebar.
9-9. Discuss the advantages and limitations of epoxy-coated rebar application in concrete structures based on past studies and experiences.
9-10. What is galvanization of rebar? List some benefits of using galvanizing reinforcing bar in concrete.
9-11. What is hot-dip galvanizing? Describe the hot-dip galvanizing process including a description of the coating structure formed on steel.
9-12. What are the processes by which galvanization protects steel from corrosion? Briefly describe each of them.
9-13. List and discuss some important physical characteristics of hot-dip galvanized coating and the advantages they provide over epoxy coating.
9-14. Discuss the various aspects of design and construction considerations for using galvanized rebar in concrete.
9-15. Discuss the factors affecting the thickness of zinc coating on rebar. What are the thickness requirements in different situations?
9-16. List and briefly discuss the tests to be conducted to ensure the quality of galvanization of rebar.
9-17. Discuss the concept of dual-coated rebar.
9-18. What is stainless steel-clad rebar? How does it compare to stainless steel rebar in terms of benefits? Write the performance characteristics of stainless steel-clad rebar.
9-19. What is chemical reactive enamel coating on rebar? Describe its applications and limitations.

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10 Non-Metallic Reinforcement

10.1 FIBER-REINFORCED POLYMER (FRP)

Fiber-reinforced polymer (FRP), also fiber-reinforced plastic, is a composite material made of a polymer matrix reinforced with fibers. The load is carried by the fibers, and the resin serves to bind them together, transfer the load to the fibers, and protect them. The fiber and volume fraction significantly affect the strength and stiffness of the FRP, while the resin type affects the failure mechanism and the fracture toughness. The fibers are usually glass, carbon, aramid, or basalt, although other fibers, such as paper, wood, or asbestos, have sometimes been used. The polymer is often an epoxy, vinyl ester, or polyester thermosetting plastic; nevertheless, phenol formaldehyde resins are still used. FRPs are widely employed in the aerospace, automotive, marine, and building sectors. The concept of FRP is schematically shown in Figure 10.1 while actual scanning electron micrographs of FRP samples are shown in Figure 10.2.

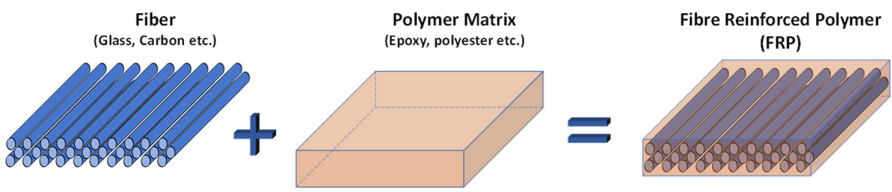

FIGURE 10.1 Concept of fiber-reinforced polymer (FRP).

Man-made or naturally occurring composite materials are made up of two or more constituent elements with significantly different physical or chemical properties that remain distinct and separate within the finished structure. Most composites have strong, stiff fibers in a weaker and less stiff matrix. The objective is usually to make a strong and stiff component, often with a low density. Commercial material commonly has glass or carbon fibers in matrices based on thermosetting polymers, such as epoxy or polyester resins. Because they are moldable after initial manufacture, thermoplastic polymers may be selected. There are other composite classes where the matrix is metal or ceramic. The majority of these are still in the development stage, with issues such as high production costs having to be solved. Furthermore, the reasons for adding fibers (or, in some cases, particles) to these composites are frequently complicated; for example, increases in creep, wear, fracture toughness, thermal stability, and so on may be desired.

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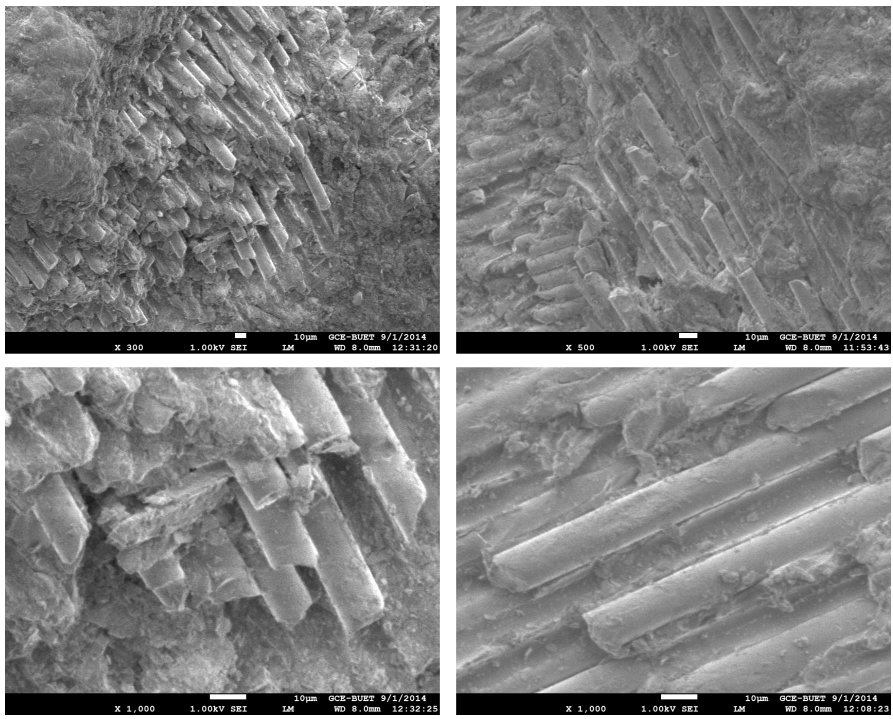

FIGURE 10.2 Typical scanning electron micrographs of GFRP composites.

10.2 BACKGROUND

Steel bars were basically the only option for reinforcing concrete structures until a few decades ago. The use of steel bars and concrete is advantageous to both parties. Steel bars can withstand tensile pressures, and concrete can withstand compression while also protecting the reinforcing steel from corrosion owing to its alkalinity.

Combinations of chlorides (de-passivation of steel) and CO_2 (carbonation of concrete) in the presence of moisture produce corrosion of the steel reinforcement. This condition causes concrete degradation and, ultimately, the structure's inability to be in service (Ahmad 2003). The degradation of various RC structures caused by chloride-ion-induced corrosion of the internal steel reinforcement became a major worry in the second part of the twentieth century. Various solutions were investigated for applications in aggressive corrosion environments (Plecnik and Ahmad 1988). Galvanized coatings, electrostatic spray fusion-bonded (powder resin) coatings, and polymer-impregnated epoxy coatings are among them. Eventually, fiber-reinforced polymer (FRP) reinforcing bars were considered an alternative to steel bars (ACI 440R-07, Dolan *et al.* 1999).

When the market need for electromagnetic-transparent (therefore non-ferrous) reinforcing bars developed in the late 1980s, the FRP reinforcing bar became a financially viable alternative as internal reinforcement for concrete structures. FRP

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composites are the most recent evolution of the very ancient notion of mixing two distinct materials to create a superior one. They consist of a reinforcing phase (fibers) embedded into a matrix (polymer). The individual components, fibers, and polymer, do not serve the function of structural materials by themselves but they do when put together. The fibers provide strength and stiffness to the composite and carry most of the applied load (ACI 440R-07), while the resin encapsulates them, thus transferring stresses and providing protection.

Following World War II, the necessity to meet aerospace industry needs that could not be reached by existing materials prompted researchers and scientists to seek alternative solutions. The solution was discovered by creating new material systems by inserting strong fibers in a polymeric matrix. FRP composite products provided various benefits over standard metallic materials. FRP composites were suitable for a variety of applications in various industries due to their innovative properties such as high tensile strength and modulus, lightness, corrosion resistance, electromagnetic transparency, and the ability to "engineer" their mechanical properties by changing constituent composition, fiber type, and orientation (ACI 440R-07). The aerospace industry began to use FRP composites as lightweight materials with superior strength and stiffness, which reduced the weight of aircraft structures. Later, other sectors like naval, defense, and sporting goods started using FRP composites extensively (Kaw 2005). FRP reinforcement for concrete structures has been under development since the 1960s in the United States (Bank 2006) and the 1970s in Europe (Taerwe and Matthys 1999; Fukuyama 1999). However, in the 1980s, the overall level of research, field demonstration, and commercialization became remarkable (Bakis *et al*. 2002).

10.3 FRP REINFORCED CONCRETE

Laboratory tests over the last two decades have shown that FRP bars can be used successfully and practically as internal reinforcement in concrete structures. The role of industry/university cooperative research became key in transferring the use of internal FRP reinforcement for concrete from the laboratory to the field. To date, reinforcing bars made of FRP have gained acceptance as internal reinforcement in concrete structures. The mechanical behavior of FRP rebars differs from the behavior of conventional steel rebars. FRP composites are anisotropic, linear, and elastic until failure and are characterized by high tensile strength only in the direction of the fibers (Nanni 1993; Wu 1990; Tamura 1993), with no yielding. A flexural concrete member reinforced with FRP rebars generally experiences extensive cracking and large deflections before failure, which is typically sudden and catastrophic. The shear strength and dowel action of FRP rebars, as well as the bond performance, are affected by the anisotropic behavior of the bars. Furthermore, the behavior of FRP bars in compression is not as good as the one in tension. Due to the anisotropic and non-homogeneous nature of FRP, the compressive modulus is lower than the tensile one (Wu 1990; Mallick 1988). There is still little experience in the use of FRP reinforcement in compression members (columns) and for moment frames or zones where moment redistribution is required (Mirmiran *et al.* 2001).

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10.4 APPLICATIONS IN NEW CONSTRUCTION

FRP rebars are suitable alternatives to steel, epoxy-coated steel, and stainless steel bars in reinforced concrete applications if durability, electromagnetic transparency, or ease of demolition in temporary applications is sought. The majority of applications (Figures 10.3 through 10.16) utilize FRP rebars to mitigate the risk of corrosion in concrete structures that operate in aggressive marine environments or are exposed to deicing salts. The service life of these structures is strictly contingent on the durability of internal reinforcement (Bank 2006). Although their initial cost (raw material and manufacturing costs) and environmental impact (CO_2 emission during the manufacturing process) is slightly higher than that of conventional steel, the use of FRP rebars in concrete structures subjected to harsh environments generates a significant potential for extending the service life of these structures and lowering their overall life cycle cost (SINTEF 2002). Applications of this type include retaining/sea walls, port infrastructure, and dry docks (Busel 2012, 2016), bridge decks (Figures 10.3, 10.7, 10.8, and 10.9), and railings where deicing salts are used (Figure 10.5), locks, and dam weirs. The use of FRP rebars is also desirable for floors (Masterbrick 1996) that host equipment sensitive to electromagnetic fields, such as magnetic resonance imaging (MRI) units, for bases of large motors or railway systems (Busel 2016).

FIGURE 10.3 Bridge deck reinforced with FRP bars (FHWA 2022, available at https://www.fhwa.dot.gov/bridge/composite/images/imgfrp002.jpg,accessed23Sept.2024).

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FIGURE 10.4 FRP dowel bars in concrete pavements.

FIGURE 10.5 GFRP reinforcement in bridge railing before casting [©Matta *et al.* 2007, reproduced with permission].

Furthermore, FRP reinforcement is ideal for temporarily reinforcing concrete structures, such as "soft-eyes" that have to be partially demolished by tunnel boring machines (TBMs). The "soft-eye" consists of a reinforcing cage using GFRP bars, which can be easily cut by the TBM (ATP 2022). FRP rebar also finds its use in constructing tunnel segments (Figure 10.6).

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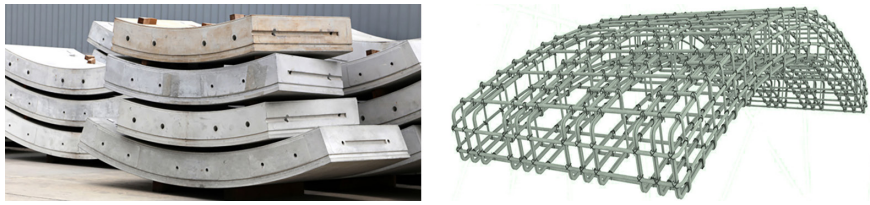

FIGURE 10.6 GFRP reinforcement cage for a tunnel segment.

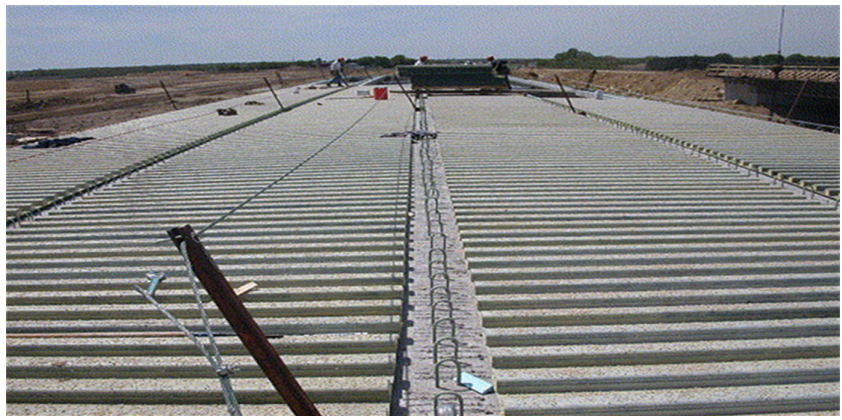

FIGURE 10.7 FRP deck panels over concrete girders [©Berg *et al.* 2006, reproduced with permission].

10.5 APPLICATIONS IN REPAIR AND RETROFITTING

FRP materials are extensively used to repair and retrofit concrete and other structures. Owing to their lighter weight, high strength, and durability, these materials, especially the CFRP, have found their way to almost every type of repair and retrofitting work.

Retrofitting applications are roughly categorized into two groups (Bank 2006). One type is strengthening, which involves increasing the strength or ductility of the original structure (usually, its deformation capacity) beyond the loads (or deformations) for which it was originally built. This increase may be required to make the structure compliant with existing building codes (especially in the case of seismic retrofitting), or it may be desirable owing to changes in the structure's usage. FRP retrofitting to increase a structure's performance (load carrying and ductility) when subjected to blast and impact loads has gained attention. The other sort of FRP retrofitting is repair. The FRP composite is utilized in this situation to retrofit an existing and deteriorating structure to restore its load-carrying capacity or ductility to the loads or displacements for which it was built (and hence is, in fact, a type of strengthening). When the original structure has deteriorated owing to environmental influences, such as corrosion of steel reinforcement in concrete, or when the original

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FIGURE 10.8 CFRP rebar application to a concrete deck [Photo ©Michigan DoT, reproduced with permission].

FIGURE 10.9 Pouring and vibration of concrete at a FRP reinforced bridge deck [©Berg *et al.* 2006, reproduced with permission].

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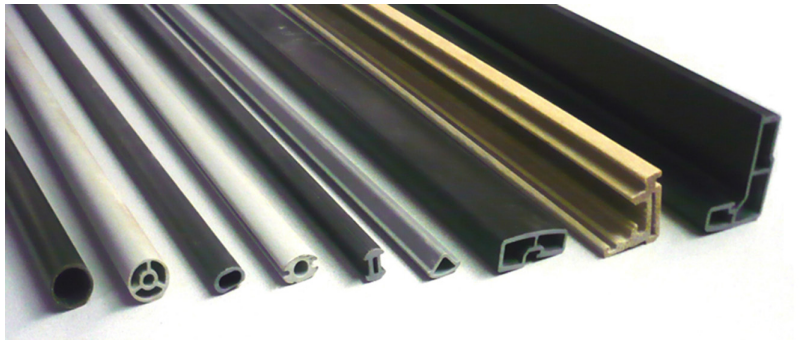

FIGURE 10.10 Pultruded shapes (Photo © Thomastechnik, Wikimedia Commons CC by SA 3.0, available at https://commons.wikimedia.org/wiki/File:Pultrusionsprofile.png, accessed 17 Sept. 2024).

structure has been damaged in service or was not built according to the original design, a repair is required. FRP retrofitting has gained popularity for improving a structure's performance (load carrying and ductility) when subjected to blast and impact loads. Repair is another type of FRP retrofitting.

In this case, the FRP composite is used to retrofit an existing and failing structure in order to restore its load-carrying capacity or ductility to the loads or displacements for which it was designed (and hence is, in fact, a type of strengthening). Repair is necessary when the original structure has deteriorated due to environmental factors such as corrosion of steel reinforcement in concrete or when the original structure has been damaged in service or was not built in accordance with the original design.

FRP retrofitting has proven to be effective on reinforced concrete structures, prestressed concrete structures, wood structures, and masonry and metal structures (Bank 2006). The primary objective of this book is to discuss reinforcing material for concrete structures. Therefore, the focus of this historical study is aimed on discussing the application of FRP retrofitting methods for reinforced concrete and prestressed concrete structures.

There are three methods of attaching FRP composite materials to concrete structures (and to masonry, timber, and even metallic structures) for retrofitting purposes. These methods are briefly discussed below.

10.5.1 Retrofitting by FRP Strips

This method makes use of premanufactured rigid FRP strips (100 mm wide and 1.4 to 1.6 mm thick) that are adhesively bonded to the structural member's surface. This method is effective in strengthening works of RC structures. The step-by-step procedure involves a) dry sandblast cleaning of the concrete surface, b) applying a thin coat of epoxy primer to the concrete surface, c) after curing/hardening of the primer, mixing the two-part epoxy (resin and hardener) and applying about 1.0 mm~1.5 mm thick layer on the primer and d) immediately attaching the strip on the epoxy and

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FIGURE 10.11 Strengthening the deck slab of Bangabandhu Bridge using CFRP Strips.

firmly attaching it on the adhesive with a roller. A few examples of such CFRP retrofitting works are shown in Figures 10.11 and 10.12.

10.5.2 Retrofitting by Wrapping with FRP Fabric

This method, also known as the hand layup method, involves forming the FRP composite on the surface of the structural member in situ using flexible dry fiber fabrics or sheets with widths ranging from 150 to 1,500 mm and liquid polymers. The procedure is similar to that of using the CFRP strips. However, depending on the design, several layers of FRP cloths can be attached to the concrete surface. A few examples of such CFRP retrofitting works are shown in Figures 10.13 and 10.14.

10.5.3 Near Surface Mounting (NSM) Method

A new variant of the premanufactured strip method called near surface mounting (NSM) has been developed in recent years. In this method, a thin, narrow FRP strip (3 by 18 mm) or small-diameter round FRP bar (6~12 mm) is inserted and then bonded adhesively into a machined groove at the surface of the concrete member. The NSM method of strengthening a beam is schematically shown in Figure 10.15. While the method is technically feasible, cutting grooves of appropriate geometry and size on concrete surfaces may be difficult, time-consuming, noisy, and costlier than the other methods of attaching FRP to concrete and require more skill. Therefore, the practical feasibility of adopting this method should be carefully judged and compared with other approaches before its adoption for a particular repair work.

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FIGURE 10.12 Strengthening the flexural capacity of a beam using CFRP Strips.

FIGURE 10.13 Enhancing concrete confinement on columns using CFRP Fabric.

10.5.4 Other Retrofitting Applications

FRP material's versatility in retrofitting concrete structures has resulted in many diversified applications. For example, GFRP bar has been used as reinforcement in the concrete overlay to repair a damaged vertical wall of a dry dock (Busel 2016).

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FIGURE 10.14 Strengthening the flexural capacity of beams using CFRP Fabric.

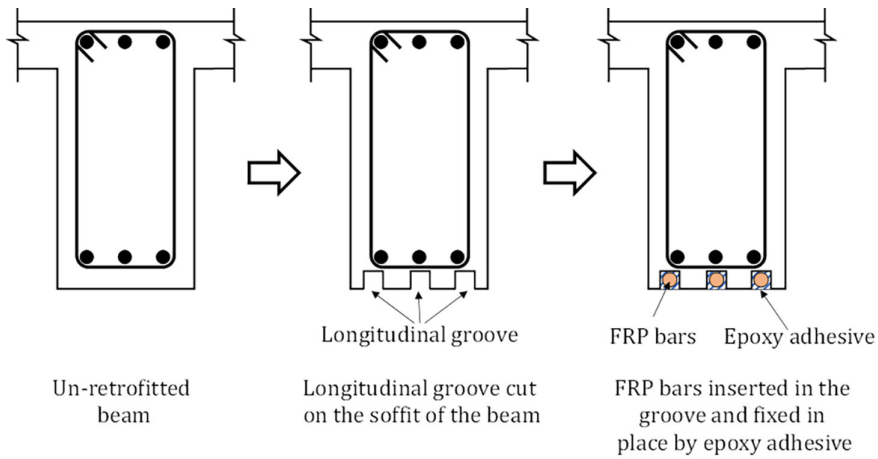
Longitudinal groove
FRP bars
Epoxy adhesive
Un-retrofitted beam
Longitudinal groove cut on the soffit of the beam
FRP bars inserted in the groove and fixed in place by epoxy adhesive

FIGURE 10.15 Concept of beam strengthening by the near surface mounting method.

Use of FRP rebar to strengthen masonry walls has been demonstrated by Coccia *et al.* (2020) as well as Majumder *et al.* (2021). Their work demonstrated how FRP rebars can be used to retrofit a masonry wall against out-of-plane vulnerability against the seismic effect. A study of similar nature was conducted by Formisano *et*

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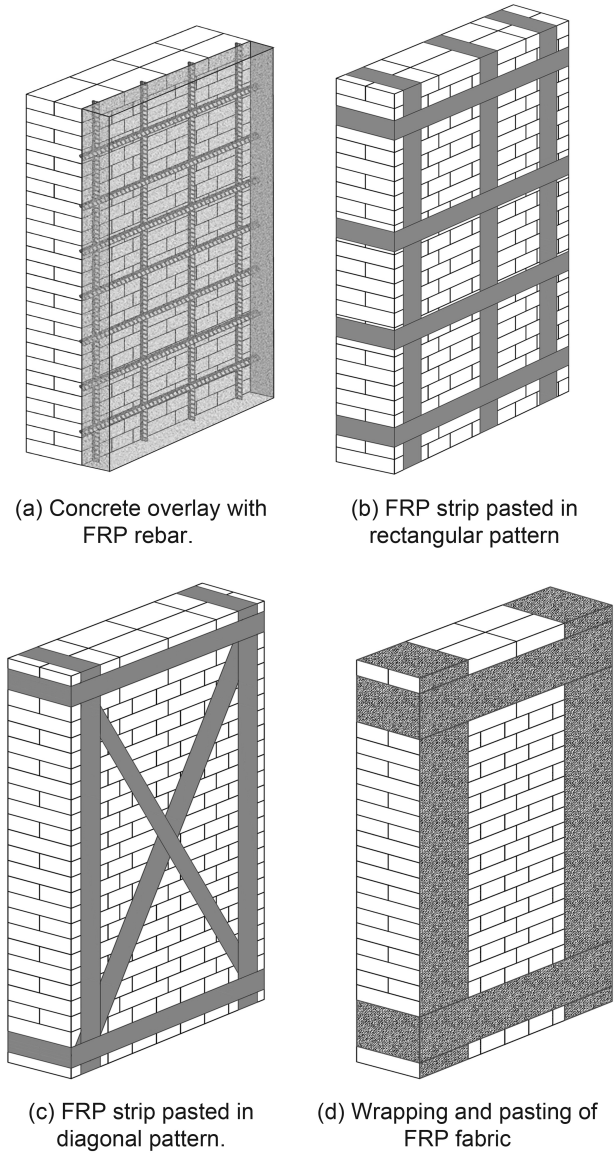
(a) Concrete overlay with FRP rebar.
(b) FRP strip pasted in rectangular pattern
(c) FRP strip pasted in diagonal pattern.
(d) Wrapping and pasting of FRP fabric

FIGURE 10.16 FRP retrofitting schemes of masonry wall.

al. (2015), who also came up with the conclusion that FRP can be an effective material to strengthen masonry walls against seismic actions. A few probable retrofitting schemes are shown in Figure 10.16.

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10.6 CONSTITUENT MATERIAL – FIBERS

FRP bar is anisotropic and may be produced utilizing several processes such as pultrusion, braiding, and weaving. Other aspects impacting the bar's qualities include fiber orientation, resin curing rate, manufacturing method, and quality control throughout manufacture. This section introduces and briefly discusses the fibers and resin matrices most typically used to create FRP bars. The properties of fiber and matrices listed here are to be considered as generic. Fibers commonly used are glass, carbon, aramid, and basalt. Glass fibers offer an economical balance between cost and specific strength; this makes them preferable to carbon and aramid in most reinforced concrete (RC) applications. Basalt fibers have recently emerged as an alternative to glass fibers. Typical properties of glass, carbon, aramid, and basalt fibers are listed in Table 10.1.

TABLE 10.1
Typical properties of fibers (single filament) (Nanni *et al.* 2014)

Type of fiber	Density (kg/m³)	Tensile strength (N/mm²)	Tensile modulus (ksi)	Ultimate tensile strain (%)	CTE (10^{-6}/°C)	Poisson's ratio
E-glass	2500	3450	10500	2.4	0.27	0.22
S-glass	2500	4550	12400	3.3	0.155	0.22
AR-glass	2253.4	1790~3445	10100~11000	2.0~3.0	N/A	N/A
High-modulus carbon	1950	2480~3996	50700~94300	0.5	–0.065 ~ –0.106	0.2
Low-modulus carbon	1750	3493	34800	1.1	–0.0324~-0.1062	0.2
Aramid (Kevlar 29)	1440	2760	9000	4.4	–0.1062 long. (1.7 radial)	0.35
Aramid (Kevlar 49)	1440	3617	18000	2.2	–0.1062 long. (1.7 radial)	0.35
Aramid (Kevlar 149)	1440	3445	25400	1.4	–0.1062 long. (1.7 radial)	0.35
Basalt	2500	700	12900	3.1	0.432	N/A

10.6.1 Glass Fiber

Fiberglass is a term used to describe a range of items created from individual glass fibers that have been assembled into various shapes. Glass fibers are split into two classes based on their geometry – continuous fibers used in yarns and textiles and discontinuous (short) fibers used in insulation and filtration batts, blankets, and boards. Fiberglass, like wool or cotton, may be spun into yarn and woven into fabric, which is occasionally used for draperies. Fiberglass textiles

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are frequently used to strengthen molded and laminated plastics. Fiberglass wool is a thick, fluffy substance comprised of discontinuous fibers that are used for sound absorption and thermal insulation. It can be found in ship and submarine bulkheads and hulls, automobile engine compartments and body panel liners, furnaces and air conditioning units, acoustical wall and ceiling panels, and architectural partitions (Nanni *et al.* 2014). Examples of glass fiber rebar is shown in Figure 10.17.

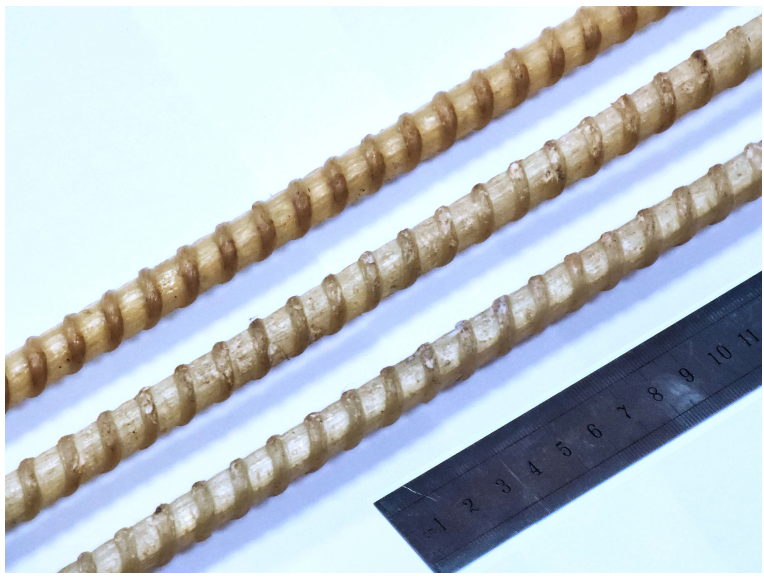

FIGURE 10.17 Glass fiber reinforced polymer rebar.

10.6.2 Carbon Fiber

Carbon-fiber-reinforced polymer, often known as carbon-fiber-reinforced plastic (CFRP or CRP, or simply carbon fiber), is a strong and lightweight fiber-reinforced polymer containing carbon fibers. When polyacrylonitrile fibers (PAN), pitch resins, or rayon are carbonized (by oxidation and thermal pyrolysis) at high temperatures, carbon fibers are formed. The strength or flexibility of the fibers can be improved by further graphitizing or stretching operations. Carbon fibers are produced in diameters similar to glass fibers, ranging from 9 to 17 micrometers. For transportation and subsequent manufacturing operations, these fibers are wrapped into bigger threads. Examples of rebar made of CFRP are shown in Figure 10.18.

10.6.3 Aramid

Aramid fibers were used to produce first-generation FRP prestressing tendons in the 1980s in Europe and Japan; however, few manufacturers still produce aramid

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FIGURE 10.18 Carbon-fiber-reinforced polymer rebar.

fiber FRP reinforcing bars or tendons. Aramid fabrics are often used to wrap columns in FRP strengthening applications and as sparse-volume weft (fill) strands in unidirectional glass or carbon fabrics. Aramid fibers consist of aromatic polyamide molecular chains. They were first developed and patented, by DuPont in 1965 under the trade name Kevlar.

A combination of their relatively high price, difficulty in processing, high moisture absorption (up to 6% by weight), low melting temperatures (around 425°C (800°F)), and relatively poor characteristics in compression have made them less suitable for FRP parts for structural engineering applications. They are used in a variety of industrial products, either as bare fabric or as reinforcements for FRP composites that require energy absorption, such as bulletproof vests (body armor), helmets, and automobile crash attenuators, because of their great tenacity and durability. They have a striking yellow hue and cost about the same as carbon fibers. Like carbon fibers, they have a negative coefficient of thermal expansion in the fiber's longitudinal direction. They are the lightest of the high performance fibers, having a density of around 1.4 g/cm^3 (0.051 lb/in^3).

10.6.4 Basalt Fibers

Basalt fiber is slightly stronger and stiffer than E-glass, environmentally safe, nontoxic, non-corrosive, and non-magnetic, and has high-heat stability and insulating characteristics (Sim *et al.* 2005; Ludovico *et al.* 2010). Although basalt fiber is manufactured with the same technology utilized for E-glass fiber, its production process requires less energy, and the primary raw material (basalt rock) is available all around the world. Basalt fiber may offer the opportunity to engineer its mechanical properties by modification of its chemical composition resulting in a fiber having an

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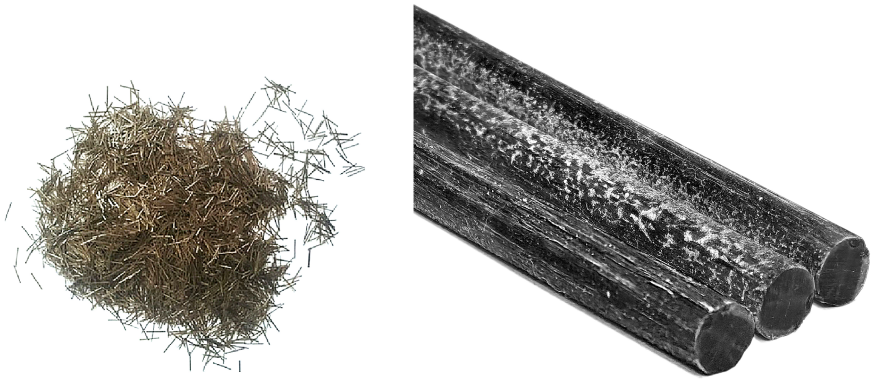

(a) Basalt fiber (Photo © Gao, L., Hu, G., Xu, N., Fu, J., Xiang, C. and Yang., C., Wikimedia Commons CC by 4.0, available at https://commons.wikimedia.org/wiki/File:Basalt-fiber.jpg, accessed 16th Sept. 2024).

(b) Basalt rebar.

FIGURE 10.19 Basalt fiber and rebar.

elastic modulus higher than that of E-glass and, at the same time, high bio-solubility. The latter property represents the capability of basalt fiber to dissolve in the long term when in contact with biological liquids. Bio-solubility is a requirement that recent international directives have introduced in the glass fiber industry and its fulfillment is considered the principal element to guide future market demand (Deak and Czigany 2008; Mingchao *et al*. 2008). Basalt fibers and concrete rebar made of basalt fibers are shown in Figure 10.19.

10.7 CONSTITUENT MATERIAL – MATRICES

Matrices are commonly thermoset polymeric resins. In their initial form, thermo-set resins are usually liquids or low melting-point solids and they are cured with a catalyst and heat, or a combination of the two. Unlike thermo-plastic resins, once cured, solid thermoset resins cannot be converted back to their original liquid form or reshaped. Epoxies, polyesters, and vinyl esters are the most common thermo-setting resins used in the composites industry. Additives and fillers may be mixed with the resin to impart performance improvements, tailor the performance of the composites, and reduce costs. Typical properties of epoxy, polyester, and vinyl ester resins are listed in Table 10.2.

10.7.1 Epoxies

Epoxy is a type of polymer that is also referred to as polyepoxide. It is used to make scratch-resistant protective coatings, fillers, and adhesive materials that can be used in a wide variety of applications. When it is in liquid form, epoxy resin has a viscous

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TABLE 10.2
Properties of different matrices

Type of resin matrix	Density (kg/m³)	Tensile strength (N/mm²)	Longitudinal modulus (N/mm²)	Poisson's ratio	CTE (10^{-6} /°C)	Moisture content (%)	Glass transition temp. (°C)
Epoxy	1190~1425	35~105	2070~3450	0.35~0.39	2.88~5.4	0.15~0.60	95~175
Polyester	1190~1425	48~130	2760~4130	0.38~0.40	2.34~1.8.	0.08~0.15	70~100
Vinyl ester	1125~1365	69~76	3000~3445	0.36~0.3	1.5~2.7	0.14~0.30	70~165

consistency; it hardens in a short amount of time – usually in less than 24 hours. It attaches to a wide variety of substrate materials, including wood, metal, glass, concrete, and stone, and it cures at room temperature. Epoxy, in its most fundamental form, is made up of a liquid epoxy resin and a liquid chemical hardener that transforms the resin into a solid and hard plastic once it has been cured. When it reaches its final solidified state, epoxy exhibits remarkable strength, dimensional stability, and resistance to chemicals.

Epoxies come in a variety of forms such as liquid, semi-liquid or in powdered form. In liquid and semi-liquid form, the resin and the hardener come in separate containers. These are thoroughly mixed in the manufacturer's recommended proportion just before the application. After the application is complete, it is kept undisturbed for a few hours to cure and harden. The majority of the applications of epoxy are done in this manner. In the powdered form, both the dry resin and the hardener generally come mixed together. Upon heating, the resin and the hardener are fused together and bonded to the surface upon which the epoxy is applied. Epoxy-coated mild steel rebar is manufactured in this manner.

The principal advantages of epoxy resin are high mechanical properties, ease of processing, low shrinkage during cure, and good adhesion to a wide variety of fibers. Epoxies have high corrosion resistance and are less affected by water and heat than other polymeric matrices. Their disadvantages are a high cost and a longer curing period (a posturing process is generally required). Epoxy resin can also be formulated with different materials or blended with other resins to achieve specific performance features. Epoxies are primarily used for fabricating high-performance composites with superior mechanical properties, resistance to corrosive liquids and environments and superior electrical properties, and good performance at elevated temperatures. Epoxy resins are compatible with glass, carbon, aramid, and basalt fibers. However, their use in the pultrusion industry is limited.

Disadvantages of epoxy include its susceptibility to disintegration under sunlight. The ultraviolet component of sunlight causes gradual chemical disintegration of the epoxy. Therefore, all epoxy products must be protected from direct sunlight.

The Glass Transition Temperature (T_g) is one of the most important properties of any epoxy and is the temperature region where the polymer transitions from a hard, glassy material to a soft, rubbery material. As epoxies are thermosetting materials

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and chemically cross-linked during the curing process, the final cured epoxy material does not melt or reflow when heated (unlike thermoplastic materials), but undergoes a slight softening (phase change) at elevated temperatures. Therefore, in any application of epoxy, the temperature of the application should be kept below the glass transition temperature, T_g, to ensure the integrity of the application.

10.7.2 Polyesters

The main advantage of polyester resins is a balance of good mechanical, chemical, and electrical properties, dimensional stability, cost, and ease of processing. Polyester resins are generally relatively inexpensive and offer good mechanical and electrical performance. Because polyesters can be chemically tailored to meet the requirements of a wide range of applications, a number of specialty polyesters are available that address specific performances such as flexibility, electrical insulation, corrosion resistance, heat and ultraviolet light resistance, fire retardancy, and optical translucence, etc. Styrene is usually mixed in large quantities (more than 10% by mass of the polymer resin) to give a low viscosity liquid. Their use in the manufacturing of FRP bars is discouraged because of lower chemical resistance as compared to vinyl esters.

10.7.3 Vinyl Esters

Vinyl esters exhibit some of the beneficial characteristics of epoxies, such as chemical resistance and high strength, as well as those properties of polyester, such as viscosity and fast curing. Vinyl esters exhibit good alkali resistance and have good wet-out and good adhesion with glass fiber, which makes them the preferred choice for manufacturing GFRP (glass FRP) composites.

10.8 MANUFACTURING BY PULTRUSION

The FRP bar is typically manufactured by pultrusion or variations of this process. Pultrusion is a continuous molding process that combines fiber reinforcement and thermosetting resin. This process is ideal for the continuous fabrication of composite parts that have a constant cross-sectional profile, such as bars. Glass fibers, initially packaged in roving, are drawn through a resin bath where the material is thoroughly impregnated with a liquid thermoset resin. Before entering the resin bath, fibers are spread out to allow for even wetting. The resin-impregnated fibers are first guided through a metal die that defines the size of the final bar and, then enter a curing oven. Before entering the oven, sand coating and helicoidal wraps are applied to the surface of the bar. Inside the oven, heat is transferred under precise temperature control to the bar. The heat activates the resin curing, changing it from a liquid to a solid. The solid bar emerges from the curing oven to the exact size of the die cavity. The bar is continuously pulled and, finally, cut to the desired length. The duration of the process varies with the size of the final bar; typically, the production speed is 3 ft (0.91 m) per minute.

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A pultruded component might have an open cross-section, like a plate or a wide-flange profile, a single closed cross-section, like a hollow tube, or a multicellular cross-section, like a panel with internal webs. The thickness of the cross-section need not be uniform throughout. Although there is a large amount of flexibility in the shape, thickness variation, and size of the part's cross-section, the cross-section must be uniform along its length. In addition, the component cannot be cured into a curved shape; it must be straight. Any length of pultruded material may be manufactured, and if it is flexible enough, it can be wound onto a spool for shipment (such as a thin FRP strip for strengthening or a small-diameter FRP rebar). Non-constant cross sections and the production of curved pieces have led to the development of non-routine pultrusion process modifications. In some forms, a core substance is employed to fill the cavity of a cellular portion.

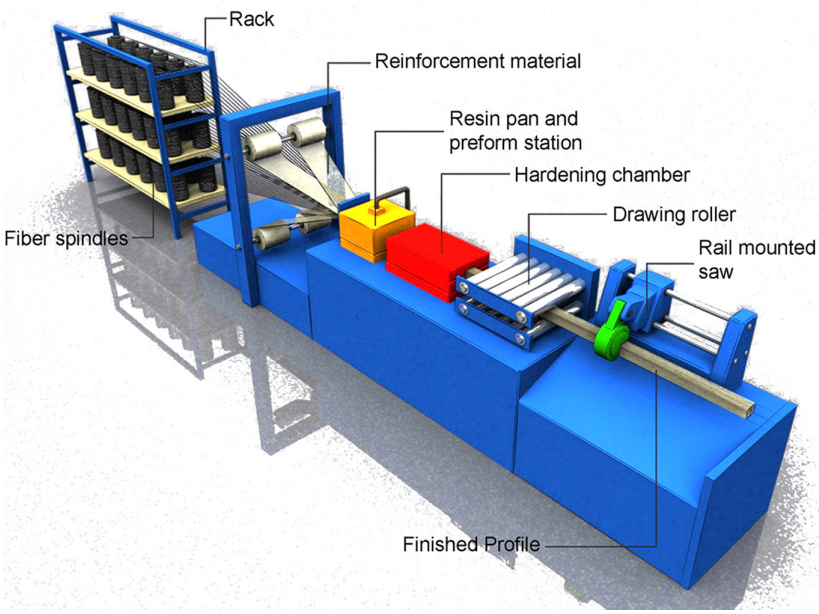

FIGURE 10.20 Pultrusion machine configurations (Photo © Florian.Arnd~commonswiki, background and language by the author, Wikimedia Commons CC by SA 3.0, original available at https://commons.wikimedia.org/wiki/File:Strangziehverfahren.png, accessed 16 Sept. 2024).

A pultrusion line or pultrusion machine is utilized to manufacture the pultruded component. Brandt Goldsworthy invented the Glastruder, the first pultrusion machine, in the early 1950s (Goldsworthy 1954). Goldsworthy and Landgraf were granted a U.S. patent for a pultrusion machine and method for producing pultruded parts in 1959. Pultrusion machines can now be purchased from a variety of companies or constructed from scratch using readily available materials and components. Experienced pultrusion

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businesses typically design and manufacture their own pultrusion machines. A schematic of a typical pultrusion line or machine is shown in Figure 10.20.

10.9 FIBER SYSTEM FOR PULTRUSION

The fiber system used in an FRP pultruded component can consist of fiber materials of various types and structures. The raw fiber is processed and supplied either in strand form on a spool as roving or tow or as broad goods on a roll as mat, fabric, veil, or tissue. The dry fiber is fed into the pultrusion die in a specific configuration so as to locate the various different fibers and fiber forms in specific portions of the cross-section of the part. High-density polyethylene (HDPE) or Teflon-coated plates with holes and slits or vinyl tubes, through which the different types of fibers pass, direct the fibers through the resin bath and into the die. Before the resin-saturated fibers enter the die mouth, the guides assist in removing excess resin from the fibers. Individual strands and mats or fabrics are typically arranged in alternating symmetric and balanced layers in FRP profiles, giving the pultruded material a laminated or layered internal architecture.

10.9.1 Glass Fiber

Individual continuous glass filaments are bundled, typically without a twist, into multifilament strands known as rovings, which are used in the pultrusion process either as is or in roving-produced fabrics. Typical scanning electron micrographs of glass fiber reinforced composites are shown in Figure 10.2. Traditionally, in the United States, roving quantity is measured in yield units (m/kg). The yields of roving are 113, 125, 228, 453, 504, 907, 925, 1310, and 1360. The number of filaments in a single roving with a given yield is dependent on the filament's fiber diameter.

A glass fiber type, known as a continuous filament (or strand) mat (CFM), is the second most widely used glass fiber product in the pultrusion industry. CFM is used in plate-like components or portions of components to provide crosswise (CW) or transverse strength and stiffness (such as the flange of a wide-flange profile). CFMs are composed of random, swirled, indefinitely long continuous glass fiber filaments held together by a polymeric binder that is soluble in resin. Due to the pulling forces exerted on the mats during pultrusion processes, chopped strand mats are typically unsuitable for pultrusion unless they are used in conjunction with a preformed combination fabric system. Continuous filament mats also aid in maintaining the position of individual rovings as they pass through the die. Due to the random orientation of the fibers in the plane of the continuous mat, it can be assumed that both the mat and the layer of cured pultruded material containing the mat have the same properties in all directions (i.e., isotropic properties) in its plane.

The range of mechanical properties of a pultruded composite consisting solely of rovings and mats is limited because unidirectional rovings provide longitudinal reinforcement and continuous strand mats provide reinforcement in all in-plane directions equally (i.e. isotropically). Fabric reinforcements, in which fibers are oriented

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in specific directions and at specific volume percentages to the pultrusion axis, can be used to obtain a wider range of properties and to "tailor" the layup (or fiber architecture) of the pultruded composite to yield specific structural properties. In the hand layup method, multiaxial plies (or layers) are common; however, multiaxial fabrics have only recently been successfully used in the pultrusion process.

10.9.2 Carbon Fiber

Carbon fiber strands, also known as tows (Figure 10.21), have been used in the pultrusion process since the 1970s to pultrude small specialized items like archery arrows and solid rods. Due to their high cost compared to glass fiber roving, they have not been widely used to produce pultruded profiles for structural engineering. Carbon fiber-reinforced epoxy pultruded FRP strengthening strips have significantly increased the use of carbon fiber FRP products in structural engineering over the last few years. The term TEX refers to the amount of carbon fiber in a tow. The number of individual carbon fiber filaments in a carbon fiber tow is usually 1000 to 48,000 (known as 1K to 48K size tows); however, high-yield tows up to 300,000 filaments (300K tow) have been produced in recent years.

FIGURE 10.21 Carbon fiber tow on spools (photo © texdata.com, tow added by the author Wikimedia Commons CC by SA 4.0, original available at https://commons.wikimedia.org/wiki/File:Carbon_fiber_material.jpg, accessed 16 Sept. 2024).

Carbon fiber fabrics typically come in the same weaves and stitched goods as glass fiber fabrics. They are typically produced in biaxial 0/90 and 45 bias-ply layups and range in weight from 200 to 1050 g/m^2. Fewer manufacturers produce them than glass fabrics, and they are typically produced as special orders from major specialty fabric suppliers for specific pultrusion applications. Only a few pultruded FRP products with carbon fiber fabrics for structural engineering are currently available. Fabrics made of carbon fiber are significantly more expensive than those made

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of glass fiber. Specialty manufacturers can also provide continuous filament carbon fiber mat, such as a glass CFM.

10.9.3 Hybrid Fabrics

Most specialty fabric manufacturers create custom hybrid fabrics with carbon, glass, and often aramid fibers in various orientations and weights; however, these fabrics are not used in regular FRP pultruded structural parts. These are usually more expensive, and used in one-off molding operations like layup and resin transfer molding to make more exotic composite material industrial products and sporting goods like surfboards, skis, snowboards, and racquets.

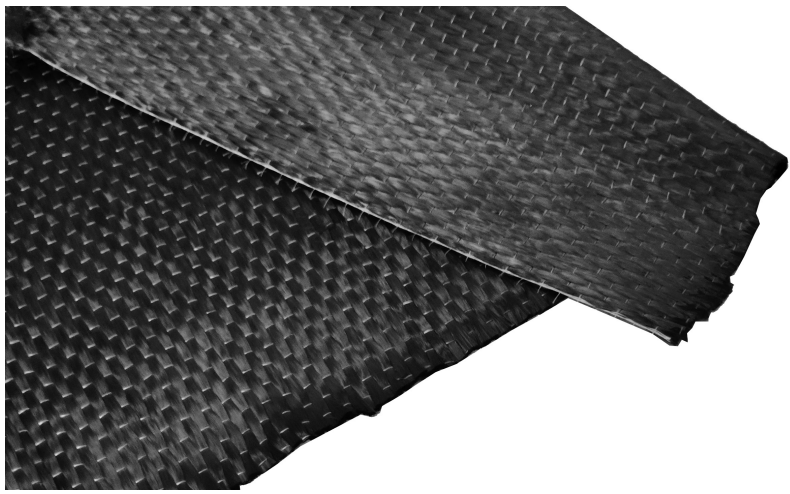

FIGURE 10.22 Carbon fiber fabric.

10.10 RESIN SYSTEM FOR PULTRUSION

Unsaturated polyesters, unsaturated vinyl esters, and epoxies are the three main thermosetting resins used in the pultrusion process. Supplementary constituents are added to each of these base resins to initiate the polymerization reaction, modify the processing variables, and customize the properties of the final FRP pultruded part. Polymerization agents, fillers, and additives are the three main categories of additional constituents. Depending on the resin type, polymerization agents are referred to as catalysts, curing agents, or even hardeners. Extenders are another name for fillers. Modifiers and process aids are other terms for additives. Meyer (1985) and Starr (2000) provide examples of pultrusion resin systems, complete with trade names and precise quantities of the various constituents used. Because most pultruders consider their resin systems to be proprietary, they rarely reveal the exact details of their resin systems. A pultrusion company usually collaborates with a resin manufacturer to develop resin systems.

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10.10.1 Polymerization Agents

Organic peroxides catalyze the reaction of unsaturated polyester and vinyl ester resins with a styrene monomer. The die heat activates the peroxide to "kick-off" or initiate the curing reaction, preventing the resin from gelling and curing in the resin bath. Different catalysts are used to cause the polymerization processes in the pultrusion die to occur at a controlled time and lengthwise position. Kicker peroxides, medium-reactivity peroxides, and finishing peroxides are the terms used to describe them (Starr 2000). Peroxides are added in amounts ranging from 0.25 to 1.5% by weight of the resin and are measured in parts per hundred (pph). During the resin mixing process, an additional styrene monomer is frequently added to the base resin (also known as the neat resin). To aid processing (by lowering resin viscosity) and cut costs, 10 to 15% by weight can be added. However, the amount of styrene in the resin mix must be carefully monitored, as unreacted styrene in the cured part can cause the part to be less durable in the environment. Pultrusion also employs blended polyester and vinylester resins to take advantage of each resin's processing and property characteristics.

10.10.2 Fillers

For three primary reasons, inorganic particulate fillers are used to fill or "extend" the base polymer resin used for pultrusion: to improve processing dynamics, reduce cost, and alter cured part properties (Lackey and Vaughan 2002). Particle sizes for inorganic fillers range from 0.5 to 8 m, with spherical or plate-like geometries. Kaolin clay (aluminum silicate), calcium carbonate, and alumina trihydrate are the three most common inorganic fillers used in pultrusion (ATH). The filler content in typical polyester, vinylester pultruded FRP profiles, and FRP rebars is between 10% and 30% by weight. Filler percentages in small pultruded parts with primarily unidirectional roving reinforcement range from 0 to 5% by weight. Pultrusions made of epoxy resin can also be filled. Small parts, like FRP strengthening strips, are rarely filled. The inorganic fillers have a density of between 2.4 and 2.6 g/cm^3, which is roughly equal to glass fiber and twice that of the resin. Fillers are significantly less expensive than resins or fibers. Alumina trihydrate serves as both a filler and a fire-retardant additive. Fillers usually reduce an FRP pultruded part's key longitudinal mechanical properties and corrosion resistance (Lackey *et al.* 1999). Fillers, on the other hand, can be used to improve properties, particularly in the transverse direction, and to change the physical properties of an FRP pultruded part (Meyer 1985). Pultruders have recently begun experimenting with nanosized particle fillers, with promising results showing FRP parts with improved corrosion resistance.

10.10.3 Additives

The third group of constituents added to the resin mix are those that aid in the processing of the cured FRP part or modify its properties (Lackey and Vaughan 2002). To keep the FRP part from sticking to the die interior, chemical release agents such as metallic stearates, fatty acids, or waxes are used. To remove entrained air from

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the resin mix, a foaming agent is frequently used. To give the finished parts different colors, pigments, or colorants, are mixed into the resin. Ultraviolet stabilizers are used to protect the resin in the cured part from the effects of sunlight. To meet code-mandated fire and flammability ratings, additives that retard flame spread in the cured FRP part, such as antimony trioxide, may be added to FRP profiles. To change the properties of the resin mix, thickening agents, toughening agents, and viscosity control agents may be added. These additives are usually added in amounts of less than 1% of the resin's weight. A low-profile or shrink additive is one additive that is frequently added in a significantly higher percentage. Low-profile additives are usually thermoplastic polymer materials that are mixed in to prevent shrinkage cracking in the interior of thick parts and on the surface of thin parts. To improve the bond between the fibers and the resin, silane coupling agents can be added to the resin mix; however, these coupling agents are usually found in the fiber's sizing. Because they all affect the resin chemistry, all of these additives can influence both the physical and mechanical properties of the FRP part (Lackey and Vaughan 2002). Even a seemingly innocuous additive like a pigment can have a significant impact on properties, and changing the color of a pultruded FRP part is not always simple.

10.11 HAND LAYUP

The manual method of constructing an FRP composite part by laying up, or rather, putting down, successive layers of fibers and impregnating them with a liquid polymer resin, which cures to form a solid FRP composite element, is known as hand layup. The solid part takes on the shape and form of the mold or surface it is applied to. The method is also known as laminating, wet layup, or simply layup, and it is used to make FRP composite laminates or panels. Hand layup is the oldest method of producing FRP parts and is used to create a wide range of FRP products. The method appears to be simple, but it takes a lot of skill and good quality control to produce a high-quality FRP part using it. Although the hand layup method used in structural engineering appears to be similar to that used in other industrial processes, there is one significant difference. When an industrial product is made, such as a sailboat hull or a tennis racquet, it is created in or on a mold and then removed after curing. In fact, to ensure that the FRP part does not stick to the mold or surface on which it was formed, a chemical or natural release agent is usually used to remove it from the molding surface. The situation is the polar opposite in structural engineering. To perform its strengthening function, the FRP composite that is applied by hand layup onto the surface of an existing structural element must be firmly adhered to or in contact with the surface. This is where the hand layup method in structural engineering becomes difficult. The method is used not only to make the FRP strengthening element but also to make the interface between the FRP element and the existing structural element. This interface, also known as the interfacial region, is a critical component of the FRP strengthening system. (When premanufactured pultruded FRP strips are used for strengthening, only the adhesive interfaces are created in the field, and the application of precured strips can be thought of as a form of hand layup, even though the actual FRP composite is not produced by hand layup.)

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10.11.1 Fiber System for Hand Layup

Two types of fiber systems are used in the hand layup method for FRP strengthening: unidirectional tow sheets and unidirectional or multidirectional woven or stitched fabrics. The most common fiber types are carbon and E-glass; however, some manufacturers also offer aramid fiber fabrics and hybrid fiber fabrics. Corrosive environments can benefit from AR-glass fiber fabrics.

Individual carbon tows, typically 12K tows, are aligned parallel to one another and held in place by an open-weave glass fiber scrim cloth and epoxy-soluble adhesive in a tow sheet. Tow sheet is the preferred term for a carbon fiber product consisting of continuous flat fibers held together by a very light scrim cloth. Unfortunately, however, this nomenclature is not universal. Some manufacturers refer to conventionally stitched fabrics (where the transverse fiber is a structural fiber and not a scrim fiber) as tow sheets and to tow sheets as fabrics. Several manufacturers also refer to glass fiber tow sheets, which are typically woven with an extremely sparse and light weft roving. As a result, rather than relying solely on verbal or written descriptions of a product, a structural engineer should always request to see a sample of the actual product being used in a structural design and listed in the project specifications.

The woven or stitched fabric type is another large family of fiber products used for FRP strengthening applications. These fabrics are similar to pultrusion fabrics, but they are typically supplied with a predominantly 0° fiber system (or a unidirectional fiber system), as shown in Figure 10.22. Individual tows in the fabric product can be clearly identified, whereas individual tows cannot be identified in the carbon tow sheet. The weave's undulating pattern can also be seen in the fabric.

10.11.2 Resin System for Hand Layup

Epoxy resins are almost exclusively used for hand layup in structural FRP applications. This is due to their superior adhesive properties and low shrinkage when cured as compared to the traditional industrial hand layup method. Epoxy resins are relatively simple to mix in the field, requiring only the mixing of one large amount of epoxy resin with another large amount of curing agent. This is in contrast to polyester and vinyl ester resins, which require very small amounts of catalysts and processing aids to be added at the time of mixing to process the resin (often less than 1%). Furthermore, unlike polyester and vinyl ester resins, which can be significantly affected by small amounts of additive or catalyst, epoxy properties are less dependent on mix ratios.

10.12 PHYSICAL AND MECHANICAL PROPERTIES OF FRP BARS

Typical physical and mechanical properties of some common types of fiber-reinforced polymer (FRP) bars are discussed as follows. From a reference perspective, the most authoritative and comprehensive source of information on the physical and mechanical properties of composites is the *Wiley Encyclopedia of Composites* (Nicolais *et al*. 2012).

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10.12.1 Density

The density of FRP bars ranges between about ⅙ to ¼ that of steel, which reduces transportation costs and makes FRP bars easy to handle at the job site (Bakis 1993; Bank 1993). Typical values of density of FRP bars are listed in Table 10.3.

TABLE 10.3
Typical FRP bar properties

		CTE				
Bar type	Density 10^3 kg/m^3	Long. 10^{-6}/°C	Transv. 10^{-6}/°C	Tensile strength, N/mm^2	Tensile modulus, N/mm^2	Rupture strain %
GFRP	2.153~3.623	0.1764~0.306	0.63~0.72	480~1580	5100~7400	1.2~3.1
CFRP	2.58~2.77	–0.342~0.0	2.16~3.06	600~3690	15900~84000	0.5~1.7
AFRP	2.153~2.437	–0.1746~–0.576	1.782~2.34	1723~2535	6000~18200	1.9~4.4

10.12.2 Coefficient of Thermal Expansion (CTE)

Longitudinal and transverse CTEs of FRP bars depend on the fiber type, resin, and volume fraction of the constituents. The polymeric matrices and the glass fibers can be considered isotropic, while carbon and aramid fibers are orthotropic. Carbon FRP (CFRP) and aramid FRP (AFRP) bars typically contract in the longitudinal direction when temperature increases and dilate when temperature decreases (negative CTEs in the longitudinal direction). Typical values of CTE of FRP bars are listed in Table 10.3.

10.12.3 Tensile Behavior

The tensile behavior of FRP bars is defined by a linear elastic stress-strain relationship up to failure (Wu 1990; Faza and GangaRao 1993). FRP bars have a higher tensile strength than steel bars but a lower ultimate tensile strain (no yielding) and a lower tensile modulus of elasticity. The longitudinal modulus of a FRP bar does not change much with its diameter, although the tensile strength does (Bakis 1993; Bank 1993). Shear lag is principally responsible for this occurrence (Volkersen 1938). Typical values of tensile properties of FRP bars are listed in Table 10.3.

Testing of FRP bars in compression is complicated by the occurrence of fiber micro buckling due to the anisotropic and non-homogeneous nature of the FRP material and can lead to inaccurate measurements (Volkersen 1938). Therefore, standard test methods are not well established yet. The compressive strength reduces by up to 45%, 80%, and 22% with respect to the values in tension for the case of glass FRP (GFRP), AFRP, and CFRP bars, respectively. The compressive modulus of elasticity is approximately 80% for GFRP, 85% for CFRP, and 100% for AFRP of the tensile modulus of elasticity for the same product (Mallick 1988; Ehsani 1993). Different modes of failure (transverse tensile failure, fiber micro buckling, or shear failure)

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may characterize the response of an FRP bar in compression, depending on the type of fiber, fiber volume fraction, and type of resin (Wu 1990; Mallick 1988; Choo *et al.* 2006).

10.12.4 Transverse Shear Behavior

The characteristics of the matrix have a significant impact on the behavior of FRP bars under transverse shear force. Transverse shear is often weak in FRP bars. Additional fibers can be braided or wound in a direction transverse to the longitudinal one to increase shear strength. FRP bars have a typical transverse shear strength of 30 to 50 MPa (FIB Bulletin 40 2007).

10.12.5 Creep Rupture

FRP bars exhibit progressive deformations when subjected to a prolonged tensile load, which may eventually lead to failure (creep rupture) after a period of time known as "endurance time." The term "endurance limit" refers to the lowest level of tensile stress below which the element will never fail, regardless of load duration. The ratio of sustained tensile stress to short-term strength, as well as the environmental variables, determine the creep-rupture endurance time: the shorter the endurance period is, the higher the sustained stress-to-strength ratio is and the harsher the environmental circumstances are (high temperature, high ultraviolet light, and high alkalinity). Smooth carbon, aramid, and glass FRP bars with a diameter of 6 mm were used in an experiment. In laboratory conditions, the bars were tested at various load levels at room temperature. The proportion of original tensile strength retained, linearly extrapolated at the 50-year endurance time, was roughly 30%, 50%, and 95% for GFRP, AFRP, and CFRP, respectively, according to test data (Yamaguchi *et al.* 1997). In comparison to GFRP bars, CFRP bars perform better under sustained loads, according to other researchers (Yamaguchi *et al.* 1997; Ando *et al.* 1997; Seki *et al.* 1997; Saadatmanesh and Tannous 1999).

10.12.6 Fatigue Behavior

Fatigue failure is not common in carbon, aramid, or glass fibers. Data on the fatigue behavior of stand-alone FRP materials have been collected during the last few decades. These results, on the other hand, came from specimens made for aeronautical purposes. The plot of the ratio of stress and initial static strength versus the logarithm of the number of cycles at failure for GFRP, AFRP, and CFRP, respectively, shows a continued downward trend of about 10%, 5–6%, and 5–8% every decade of logarithmic life (Saadatmanesh and Tannous 1999; Roylance and Roylance 1982; Mandell 1982; Curtis 1989; Mandell and Meier 1983; Odagiri *et al.* 1997). Moisture, environmental temperature, cyclic loading frequencies, and the ratio of maximum-to-minimum cyclic stress were shown to affect the fatigue strength of CFRP bars encased in concrete (Tannous and Saadatmanesh 1999; Rahman and Kingsley 1996; Adimi *et al.* 1998). In the study of El-Ragaby *et al.* (2007), full-scale GFRP

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reinforced concrete bridge deck specimens outperformed steel counterparts in terms of fatigue performance and fatigue life.

10.12.7 Durability

The environment affects the mechanical properties of fiber-reinforced plastic (FRP) bars. The tensile and bonding properties of FRP bars may be affected by the presence of water, alkaline or acidic solutions, salt solutions, ultraviolet light, and high temperatures.

There are data from short-term experiments on bare, unstressed bars exposed to alkaline environments. However, it is difficult to extrapolate these data to field conditions and predict lifetimes (Kamal and Boulfiza 2011). High pH decreases GFRP bars' tensile strength and modulus (Porter and Barnes 1998). High temperature and prolonged exposure time accelerate the degradation. The decrease in tensile strength and modulus of (stressed or unstressed) GFRP bars ranges from 0 to 75% and 0 to 20%, respectively. In the case of unstressed CFRP, it has been reported that both strength and stiffness decrease by between 0 and 20%. (Sasaki *et al.* 1997). The degradation of bars embedded in concrete at various temperatures and with good fiber-resin combinations is minimal, but it increases with temperature and stress (Coomarasamy and Goodman 1997; GangaRao and Vijay 1997). Sunlight (ultraviolet rays) and moisture have a detrimental effect on the tensile strength of fiber-reinforced plastic (FRP) bars with 0% to 40% of the initial value lost in the case of GFRP bars, 0% to 30% for AFRP bars, and 0% to 20% for CFRP bars (Takewaka and Khin 1996; Bootle *et al.* 2001). Despite the fact that FRP bars embedded in concrete are not exposed to ultraviolet rays during service, ultraviolet light may cause degradation during storage.

The Intelligent Sensing for Innovative Structures (ISIS) Canada Research Network acquired data on the endurance of GFRP bars in concrete exposed to natural settings during a field research (Mufti 2005; Mufti *et al.* 2007). Five buildings were chosen for concrete cores containing GFRP bars: a 5-year-old harbor wharf and four 6- to 8-year-old reinforced concrete (RC) bridges. At the microscopic level, the GFRP bars were examined for their physical and chemical makeup. The experimental results were compared to those obtained from control GFRP bars that had been conserved under laboratory conditions (Nkurunziza *et al.* 2005). The findings of the analyses revealed that the GFRP bars in real-life concrete constructions do not degrade. The analyses also indicated:

- No alkali ingress was observed in the GFRP bars from the concrete pore solution.
- The matrix in all GFRP bars was intact and unaltered from its original state.
- Neither hydrolysis nor significant changes in the glass transition temperature of the matrix took place after exposure for 5 to 8 years after the combined effects of the alkaline environment in the concrete and the external natural environment.

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10.12.8 Bond-to-Concrete Behavior

The design, manufacturing process, mechanical qualities of the bar, and ambient variables all influence the bond between FRP and concrete. Chemical bond (interface adhesion resistance), friction, and mechanical interlock transmit bond stresses at the FRP bar/concrete interface due to irregularity of the interface. Bond stresses in the FRP bar are passed to the reinforcing fibers via the resin. As a result, the shear strength of the resin limits the bond behavior of an FRP bar.

10.13 TEST METHODS

FRP composites, such as FRP reinforcing bars, FRP strips or wraps, and FRP profile sections, are utilized in structural engineering applications as FRP parts or components. A structural engineer uses the geometric and material properties of the members to perform iterative analytical calculations in terms of member stress resultants, such as the bending moment (M), twisting moment (T), shear force (V), and axial force (F), which are obtained from the applied loads to determine the member deformations and capacities (also known as strengths). The member deformations are calculated using the member's rigidities, which are calculated using the geometric dimensions of the member cross-section and material parameters. The material strengths and geometric parameters of the section are used to calculate the member load-carrying capacity.

The material strengths and stiffnesses of homogenous isotropic materials like steel are not position-dependent in the section, and the load-carrying capacity may be computed very readily. The stiffness and strength properties of FRP members, on the other hand, are location dependent within the cross-section on the fiber, lamina, and laminate levels due to the material's inhomogeneity and anisotropy. Because full-section rigidities and load-carrying capabilities are more difficult to describe in the form of simple stress-resultant equations, this tends to complicate the design process. In the analysis of FRP elements for structural engineering, however, several simplifying assumptions can be made, resulting in a significantly simplified design procedure. For certain sorts of computations, one common assumption is that the properties of the FRP part can be regarded as homogeneous on a full-section level. In this situation, tests on the complete cross-section (and typically the entire length) of the part yield the full-section member properties required for the structural engineering design of an as-delivered or as-produced part. Full-section tests are used to determine effective characteristics for design calculations based on the as-designed performance of the entire FRP part while in service.

Unique test procedures for FRP bars are necessary due to differences in the physical and mechanical behavior of FRP materials compared to steel. For application in structural concrete, standards development organizations have created consensus-based test procedures for FRP reinforcement. Standard test techniques for the usage of FRP composites in structural engineering are available from the American Society for Testing and Materials (ASTM) International and the International Organization for Standardization (ISO). The American Concrete Institute has also created direct

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testing methodologies. The ACI Committee 440 document "Guide Test Methods for Fiber-Reinforced Polymer (FRP) for Reinforcing or Strengthening Concrete and Masonry Structures" lays out model test methods for short- and long-term mechanical, thermo-mechanical, and durability testing of FRP bars that have not yet been standardized by ASTM. These suggested test procedures are based on study findings and publications from around the world.

10.13.1 ASTM Test Methods

Table 10.4 provides a list of the test methods applicable to FRP bars for use in non-prestressed concrete as available in ASTM. Of these methods, the three that were specifically developed for FRP bars to be used in concrete are briefly discussed here..

TABLE 10.4
ASTM test methods for FRP reinforcement

Property to be tested	ASTM test methods
Cross-sectional area	D7205 and D7205M
Shear strength	D7205 and D7205M
Creep properties	D7617 and D7617M
Flexural properties	D790 and D4476
Coefficient of thermal expansion (CTE)	E831 and D696
Glass transition temperature	E1356, E1640, D648, and E2092

10.13.1.1 Tensile Properties by ASTM D7205

The area of the cross-section of the FRP bar specimen is measured by applying the water displacement method. A bar sample of known length is immersed in a graduated cylinder filled with water. The increase of volume after immersing the sample is divided by the sample length to calculate the cross-sectional area. At least five measurements per lot of bars are required.

The failure of an FRP bar specimen with a specific diameter and a length of at least 40 diameters is tested. The cross-sectional area is divided by the tensile capacity (load at failure) to get the bar's tensile strength. The difference between the stress-strain curve values at 20% and 50% of the tensile capacity is used to compute the tensile modulus, assuming the stress-strain curve is linear within this load range. The ultimate strain can be computed from the ultimate tensile capacity and modulus of elasticity, or it can be measured when the ultimate tensile capacity is attained. Photographic views of a typical test setup and typical failure mode of a CFRP bar are shown in Figure 10.23. At least five specimens are required.

Typical tensile stress vs. strain diagrams of FRP composites are shown in Figure 10.24. For comparison, a typical stress-strain diagram of a Grade B420DWR mild steel rebar is also shown. It can be observed that FRP composites can have high to very high tensile strength compared to mild steel. However, FRP composites are

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brittle in nature having little or no ductility as indicated by abrupt rupture at the maximum stress (Figure 10.24). When utilizing ASTM D7205, measuring the mechanical characteristics of big FRP bars presents some unique issues. The challenge with typical wedge-type grips stems from the huge transverse stresses that must be applied to the bar in order to apply strong enough longitudinal tensile strains. Local stress triaxiality caused by stress concentrations near gripping sections frequently results in early failure at these locations rather than the desired test gauge section. Premature damage in the gripping zone happens commonly in unidirectional FRP composites, whose transverse strength is substantially lower than the longitudinal strength and is dependent on the gripping mechanism. Varied grip designs have different load transfer processes; however, they can be divided into two classes. The first group relies on the interface interaction to transfer load by applying a clamping force. If the clamping force is applied equally, the loaded end will acquire significant shear stress. The second group uses the standard wedge grip's underlying idea with various modifications to reduce shear stress concentrations on the laden ends.

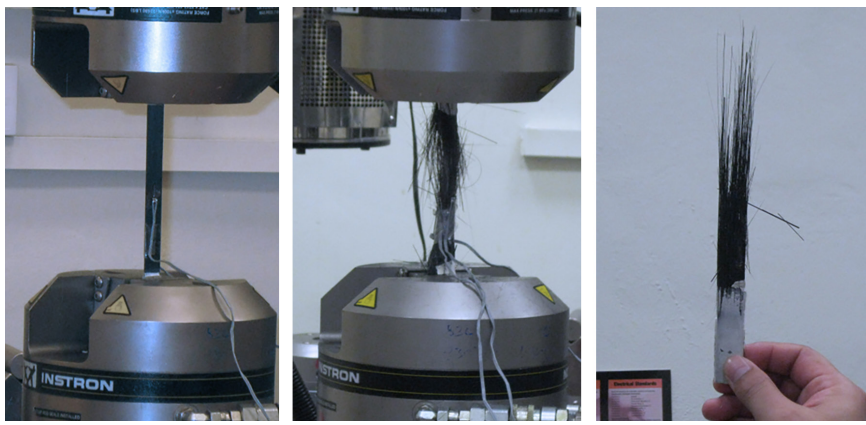

FIGURE 10.23 (a) Typical tensile test setup before failure; (b) after failure; (c) close-up view of the CFRP specimen after failure.

As shown in Figure 10.25, an ASTM-recommended system is included in Annex A1 of ASTM D7205. It uses steel pipes filled with expanding grout or polymer resin combined with sand. The anchor is designed so that the grout forms a cylindrical shell around the specimen. The expanding grout puts pressure on the specimen after curing, reducing the chances of precipitating failure in the grip zone. The specimen's clamping pressure is determined by the steel pipe's confinement and the parameters of the grout layer, such as thickness, modulus of elasticity, Poisson's ratio, and coefficient of linear expansion. The expanding grout-based grip has the advantage of being able to alter the thickness of the grout layer, the level of confinement, and the specimen embedment length (gripping length) to produce the desired gripping behavior. A precise estimate of the gripping pressure as a function of steel pipe size and grout thickness (volume) is required to develop such a versatile design.

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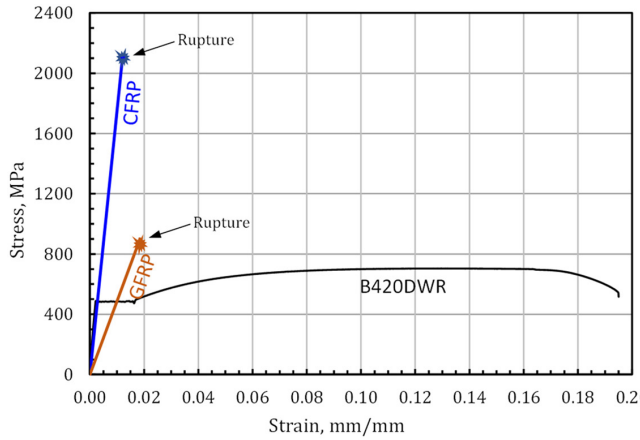

FIGURE 10.24 Typical tensile stress-strain curve of GFRP and CFRP bars.

10.13.1.2 Shear Strength by ASTM D7617

The shear strength is measured by forcing the FRP bar specimen to fail due to transverse shear. The test procedure is similar to that conducted for the shear strength of steel and other metals.

10.13.1.3 Tensile Creep Properties by ASTM D7337

At specific ages (such as after 1, 10, 100, and 1000 hours) and in conjunction with at least five predetermined levels of load, the load-induced, time-dependent tensile stresses of the FRP bar are measured. The experimental strain values are plotted against time (on a logarithmic scale), and the least-square approach is used to derive an approximated line from the graph data. The creep-rupture load ratio is the load ratio calculated from the calculated approximate line at 1 million hours. The million-hour creep-rupture capacity is the load that corresponds to this creep-rupture load ratio. Divide this load capacity by the cross-sectional area to get the million-hour creep-rupture strength. At least five specimens are required for each load level.

10.13.2 ACI 440 Test Methods

This section provides an overview of the test methods applicable to FRP bars for use in non-prestressed concrete that are outlined in ACI 440.3 *Guide Test Methods for Fiber-Reinforced Polymer (FRP) for Reinforcing or Strengthening Concrete and Masonry Structures.*

10.13.2.1 Test Methods for Bond Strength by Pullout Testing

As illustrated in Figure 10.28, many test methods for determining different bond values of FRP reinforcement in concrete structures are available (Al-Zahrani *et al.* 1996; Benmokrane *et al.* 2001; Chaallal and Benmokrane 1993; Freimanis *et al.* 1998). Pullout testing is used to determine the bond strength of the FRP bar

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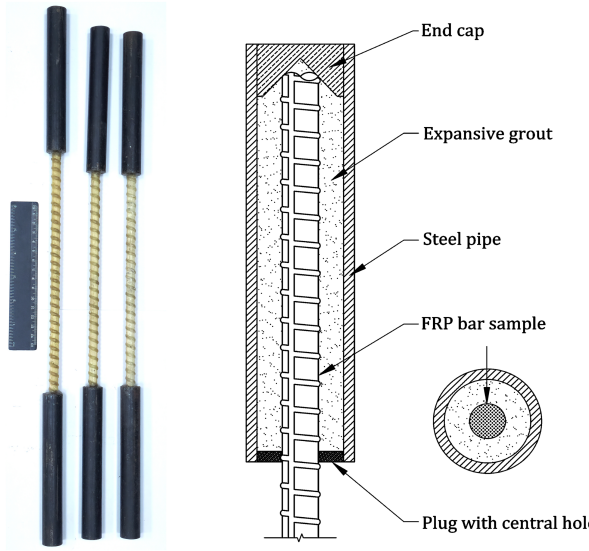

FIGURE 10.25 Steel pipe sleeve end gripping system for FRP bars.

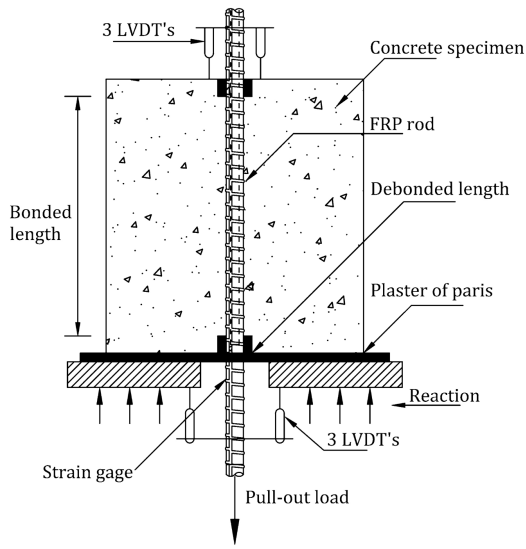

FIGURE 10.26 Pullout bond test for FRP rod (adapted from Nanni *et al.* 2014).

specimen. The direct pullout test involves testing the pullout strength of an FRP bar embedded in a concrete block over a specified bonded length. To prevent any unwanted confinement effect that may affect the bond properties, a bond breaker is put along the first five diameters on the pulling side of the bar. Figure 10.26 depicts a

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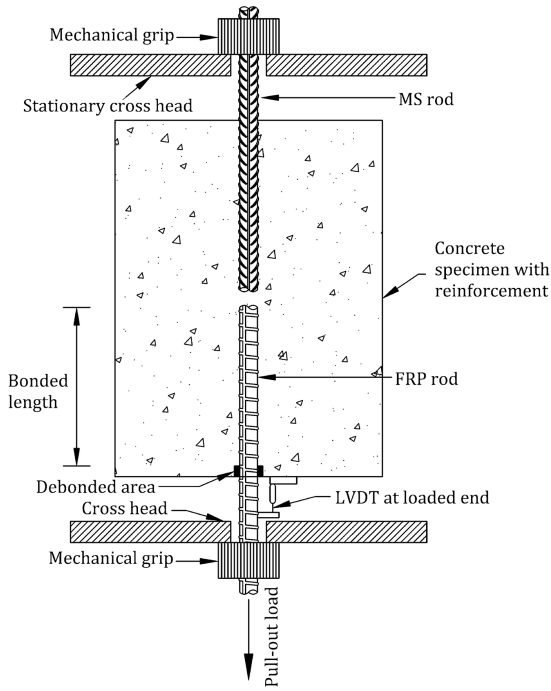

FIGURE 10.27 Rod-rod pullout test for FRP rod (adapted from Nanni *et al.* 2014).

schematic view of a typical test setup. A minimum of five test specimens should be used for each test condition. The rod-rod pullout test (Figure 10.27) and the beam-type pullout test are two more types of pullout testing (Figure 10.29).

10.13.2.2 Test Method for Strength of Bent Bars and Stirrups at Bend Locations

The straight portion of an FRP C-shaped stirrup whose bent ends are embedded in two concrete blocks is tested in tension to determine the FRP stirrup specimen's ultimate load capacity (Morphy *et al.* 1997). Two FRP C-shaped stirrups are tested simultaneously to ensure stability throughout testing, and failure is induced to occur at one of the two ends. The test setup for a FRP bent bar specimen is shown in Figure 10.30. The strength-reduction factor due to bend effects is calculated by measuring the ultimate tensile capacity of the bent FRP stirrup and comparing it to the ultimate tensile strength. A minimum of five specimens is necessary.

10.13.2.3 Accelerated Test Method for Alkali Resistance

The alkali resistance of the FRP bar specimen has been extensively studied (Benmokrane *et al.* 2002; Gentry *et al.* 1998). It is measured using three separate conditionings. After being aged in an alkaline solution (with a pH between 12.6 and 13) at 140°F (60°C) for exposure durations of 1, 2, 3, 4, and 6 months, a first set of

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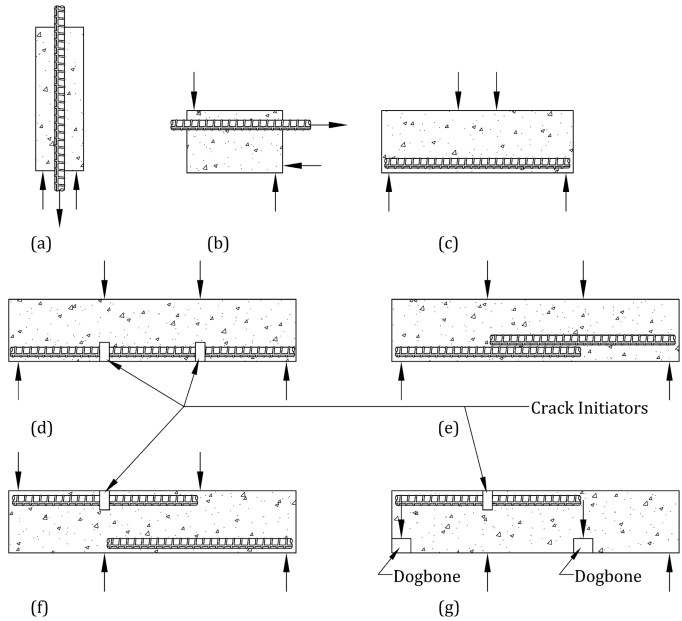

FIGURE 10.28 Types of test methods for different bond values of FRP reinforcement in concrete: (a) pullout specimen; (b) beam-end specimen; (c) simple beam specimen; (d) hinged beam-end specimen; (e) splice specimen; (f) cantilever beam specimen (without dogbones); and (g) cantilever beam specimen (with dog-bones) (adapted from ACI 440.3).

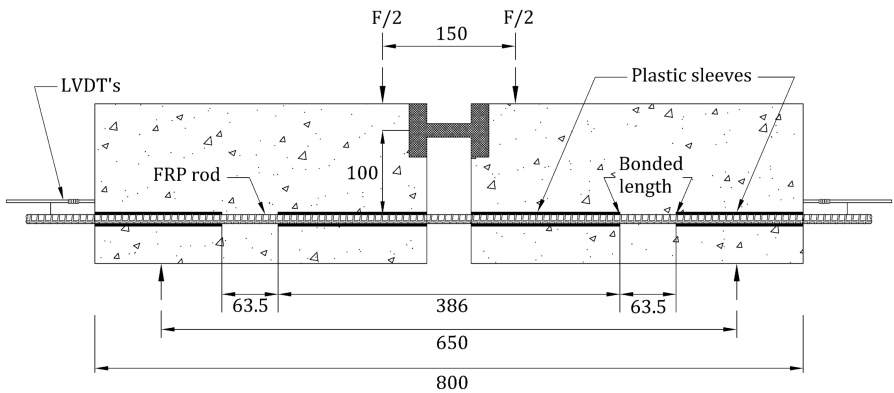

FIGURE 10.29 Beam type pullout bond test for FRP rod (adapted from Nanni *et al.* 2014).

bar specimens is tested in tension. A second set of bar specimens are subjected to a continuous tensile stress for 1, 2, 3, 4, and 6 months while immersed in an alkaline solution inside an environmental cabinet at a constant temperature of 140°F (60°C). The same conditioning is applied to a third set of bar specimens buried in concrete

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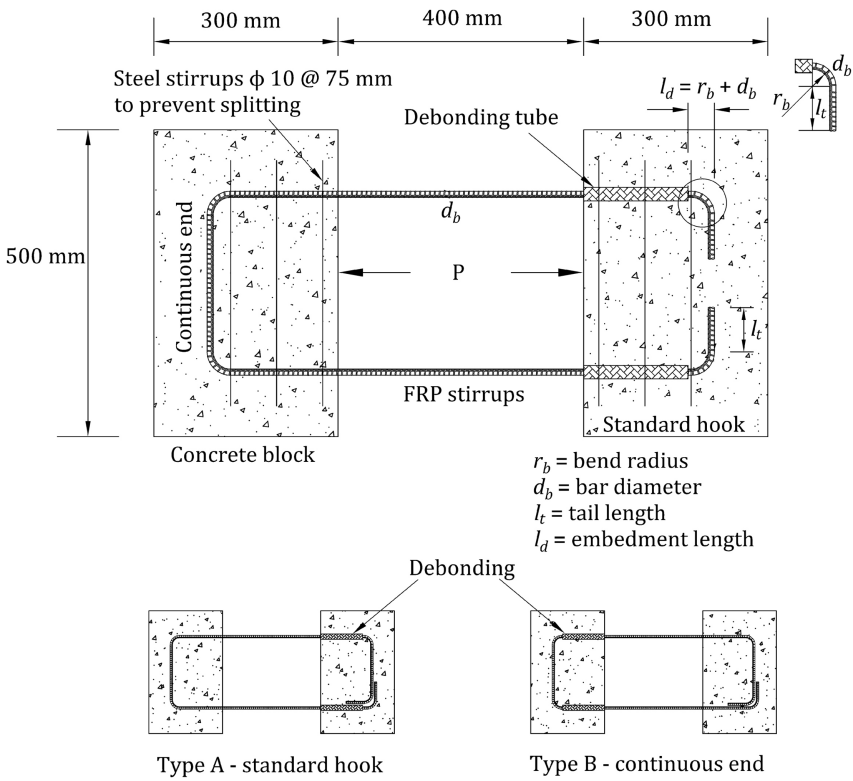

FIGURE 10.30 Schematic test setup for strength of bent bars and stirrups at bend locations (adapted from El-Sayed *et al*. 2007 and ACI 440.3).

as the second group. The size of the sustained load should be typical of service conditions or cause at least 2000 macrostrain in tensile strain. The mass change and tensile capacity retention for all specimens are computed at the end of the three series of testing. The ratio between the ultimate tensile capacity of the conditioned FRP bar specimen and the ultimate tensile capacity of a benchmark non-conditioned FRP bar is the tensile capacity retention. Each form of conditioning requires at least five samples.

10.13.2.4 Test Method for Tensile Fatigue of FRP Bars

The tensile fatigue of the FRP bar is determined by applying a tensile load with a magnitude ranging from 20 to 60% of the tensile capacity and a frequency ranging from 1 to 10 Hz. At a cycle count of between 10^3 and 2×10^6 cycles, the FRP bar specimen is projected to fail in the gauge length. The number of repeated loading cycles required to fail the FRP bar is used to produce S–N curves for a certain set of testing conditions, with the highest value of the repeated load as the primary variable. At least five specimens are required for each load level.

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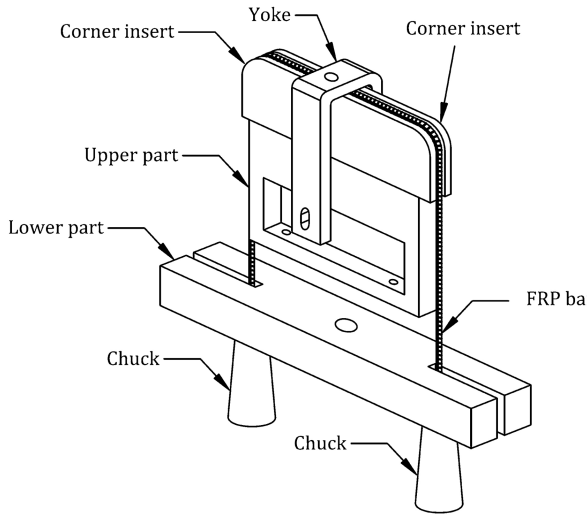

FIGURE 10.31 Schematic test setup for determining the effect of corner radius on tensile strength (adapted from ACI 440.3).

10.13.2.5 Test Method for Determining the Effect of Corner Radius on Tensile Strength

A special fixture is used to evaluate the influence of the corner radius on the tensile strength of the FRP bar. The test involves pulling the straight sides of a reverse-U shaped FRP bar while the fixture, which reacts to the testing machine frame, allows the load to be transferred to the bending portion. Figure 10.31 depicts a schematic view of the test setup. A minimum of five specimens is necessary.

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QUESTIONS

10-1. What do you understand by fiber-reinforced polymer (FRP)? Discuss with illustrations.
10-2. Provide a brief historical account of the development and use of FRP.
10-3. Briefly describe how the behavior of FRP reinforced concrete differs from conventional mild steel reinforced concrete.
10-4. Discuss possible usages of FRP in new concrete structures with examples.
10-5. Discuss usages of FRP in the retrofitting and strengthening of concrete structures with examples.
10-6. List different kinds of fiber used to produce FRP rebar. Briefly discuss each of them.

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10-7. List and briefly discuss different types of matrix material used in FRP.
10-8. What is epoxy? Discuss different processes of forming epoxy from its constituents.
10-9. What are the advantages and disadvantages of epoxy?
10-10. What is glass transition temperature of epoxy and how does it influence the application of epoxy?
10-11. Describe the pultrusion process of manufacturing FRP rebar.
10-12. List different types of fiber systems used in the pultrusion process of manufacturing FRP products.
10-13. Discuss the mechanical behavior of FRP bars in tension, compression, and shear.
10-14. Write short notes on:
    a. Creep rupture behavior of FRP.
    b. Fatigue behavior of FRP.
    c. Durability characteristics of FRP.
10-15. Discuss the tensile testing procedure of FRP samples in accordance with the ASTM D7205 standard.
10-16. What are the problems associated with wedge type grips while testing FRP in tension? What is the alternate ASTM recommended system? Discuss with neat sketches.
10-17. List different test methods prescribed by ACI to test various properties of FRP.

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Appendix

TABLE A.1
ASTM bar designations and weight (adopted from ASTM A615/A615M)

a. Standard bars

Bar designation no.	Nominal weight, lb/ft [nominal mass, kg/m]	Nominal dimensions	
		Diameter, in. [mm]	Cross-sectional area, in.² [mm²]
3 [10]	0.376 [0.560]	0.375 [9.5]	0.11 [71]
4 [13]	0.668 [0.994]	0.500 [12.7]	0.20 [129]
5 [16]	1.043 [1.552]	0.625 [15.9]	0.31 [199]
6 [19]	1.502 [2.235]	0.750 [19.1]	0.44 [284]
7 [22]	2.044 [3.042]	0.875 [22.2]	0.60 [387]
8 [25]	2.670 [3.973]	1.000 [25.4]	0.79 [510]
9 [29]	3.400 [5.060]	1.128 [28.7]	1.00 [645]
10 [32]	4.303 [6.404]	1.270 [32.3]	1.27 [819]
11 [36]	5.313 [7.907]	1.410 [35.8]	1.56 [1006]
14 [43]	7.65 [11.38]	1.693 [43.0]	2.25 [1452]
18 [57]	13.60 [20.24]	2.257 [57.3]	4.00 [2581]

b. Alternate bars

Bar designation no.	Nominal weight, lb/ft [Nominal mass, kg/m]	Nominal Dimensions	
		Diameter, in. [mm]	Cross-Sectional Area, in.² [mm²]
10	0.414 [0.617]	0.394 [10.0]	0.12 [79]
12	0.597 [0.888]	0.472 [12.0]	0.18 [113]
16	1.061 [1.578]	0.630 [16.0]	0.31 [201]
20	0.657 [2.466]	0.787 [20.0]	0.49 [314]
25	2.589 [3.853]	0.984 [25.0]	0.76 [491]
28	3.248 [3.834]	1.102 [28.0]	0.95 [616]
32	4.242 [6.313]	1.260 [32.0]	1.25 [804]
36	5.369 [7.990]	1.417 [36.0]	1.58 [1018]
40	6.629 [9.865]	1.575 [40.0]	1.95 [1257]
50	10.36 [15.41]	1.969 [50.0]	3.04 [1963]
60	14.91 [22.20]	2.362 [60.0]	4.38 [2827]

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TABLE A.2
ISO 6935-2:2015 standard bar diameters and weights (adopted from ISO 6935-2:2015)

Nominal bar diameter[a], mm	Nominal cross-sectional area[b], S_o, mm²	Mass per unit length	
		Nominal[c], kg/m	Permissible deviation[d]%
6	28.3	0.222	±8
8	50.3	0.395	±8
10	78.5	0.617	±6
12	113	0.888	±6
14	154	1.21	±5
16	201	1.58	±5
20	314	2.47	±5
25	491	3.85	±4
28	616	4.84	±4
32	804	6.31	±4
40	1 257	9.86	±4
50	1964	15.42	±4

[a]Diameters larger than 50 mm should be agreed between the manufacturer and purchaser. The permissible deviation on mass for such bars shall be ± 4%.
[b]$S_o = 0.7854 \times d^2$
[c]Mass per unit length = $7.85 \times 10^{-3} \times S_o$.
[d]Permissible deviation refers to a single bar.

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TABLE A.3
ASTM rebar deformation requirements

a. Standard bars

Bar designation no.	Perimeter, in. [mm]	Deformation requirements, in. [mm] Maximum average spacing	Maximum average height	Maximum gap (chord of 12.5% of nominal perimeter)
3 [10]	1.178 [29.9]	0.262 [6.7]	0.015 [0.38]	0.143 [3.6]
4 [13]	1.571 [39.9]	0.350 [8.9]	0.020 [0.51]	0.191 [4.9]
5 [16]	1.963 [49.9]	0.437 [11.1]	0.028 [0.71]	0.239 [6.1]
6 [19]	2.356 [59.8]	0.525 [13.3]	0.038 [0.97]	0.286 [7.3]
7 [22]	2.749 [69.8]	0.612 [15.5]	0.044 [1.12]	0.334 [8.5]
8 [25]	3.142 [79.8]	0.700 [17.8]	0.050 [1.27]	0.383 [9.7]
9 [29]	3.544 [90.0]	0.790 [20.1]	0.056 [1.42]	0.431 [10.9]
10 [32]	3.990 [101.3]	0.889 [22.6]	0.064 [1.63]	0.487 [12.4]
11 [36]	4.430 [112.5]	0.987 [25.1]	0.071 [1.80]	0.540 [13.7]
14 [43]	5.32 [135.1]	1.185 [30.1]	0.085 [2.16]	0.648 [16.5]
18 [57]	7.09 [180.1]	1.58 [40.1]	0.102 [2.59]	0.864 [21.9]

b. Alternate bars

Bar designation no	Perimeter	Deformation requirements, in. [mm] Maximum average spacing	Maximum average height	Maximum gap (chord of 12.5% of nominal perimeter)
10	1.237 [31.4]	0.276 [7.0]	0.016 [0.40]	0.151 [3.8]
12	1.484 [37.7]	0.331 [8.4]	0.019 [0.48]	0.181 [4.6]
16	1.979 [50.3]	0.441 [11.2]	0.028 [0.72]	0.241 [6.1]
20	2.474 [62.8]	0.551 [14.0]	0.039 [1.00]	0.301 [7.7]
25	3.092 [78.5]	0.689 [17.5]	0.049 [1.25]	0.377 [9.6]
28	3.463 [88.0]	0.772 [19.6]	0.055 [1.40]	0.422 [10.7]
32	3.958 [100.5]	0.882 [22.4]	0.063 [1.60]	0.482 [12.2]
36	4.453 [113.1]	0.992 [25.2]	0.071 [1.80]	0.542 [13.8]
40	4.947 [125.7]	0.102 [28.0]	0.079 [2.00]	0.603 [15.3]
50	6.184 [157.1]	1.387 [35.0]	0.098 [2.50]	0.753 [19.1]
60	7.421 [188.5]	1.654 [42.0]	0.106 [2.70]	0.904 [23.0]

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TABLE A.4
Requirement of ribs/deformation in ISO 6935-2:2015 (see Figure 2.2).

	Nominal bar diameter, d, mm	Ribs of uniform height	Crescent-shaped ribs
Rib height, a minimum	All	$0.05d$	$0.065d$
Rib spacing, c range	$6 \le d < 10$	$0.5d \le c \le 0.7d$	$0.5d \le c \le 1.0d$
	$d > 10$	$0.5d \le c \le 0.7d$	$0.5d \le c \le 0.8d$
Transverse-rib inclination, β	All	$35° \le \beta \le 90°$	$35° \le \beta \le 75°$
Transverse-rib flank inclination, α	All	$\alpha \ge 45°$	$\alpha \ge 45°$
Ribless perimeter, $\sum f_i$ Maximum	All		$0.25d\pi$

TABLE A.5
ASTM A615 tensile strength requirements

a. Standard bar sizes

	Grade 60 [420]	Grade 75 [520]	Grade 80 [550]	Grade 100 [690]
Tensile strength, min, psi	90 000	100 000	105 000	115 000
[MPa]	[620]	[690]	[725]	[790]
Yield strength, min, psi	60 000	75 000	80 000	100 000
[MPa]	[420]	[520]	[550]	[690]
Elongation in 8 in. [200 mm], min, % bar designation no.				
3 [10]	9	7	7	7
4, 5 [13, 16]	9	7	7	7
6 [19]	9	7	7	7
7, 8 [22, 25]	8	7	7	7
9, 10, 11 [29, 32, 36]	7	6	6	6
14, 18, 20 [43, 57, 64]	7	6	6	6

b. Alternate bar sizes

	Grade 60 [420]	Grade 75 [520]	Grade 80 [550]	Grade 100 [690]
Tensile strength, min, psi	90 000	100 000	105 000	115 000
[MPa]	[620]	[690]	[725]	[790]
Yield strength, min, psi	60 000	75 000	80 000	100 000
[MPa]	[420]	[520]	[550]	[690]
Elongation in 8 in. [200 mm], min, % bar designation no.				
10	9	7	7	7
12, 16	9	7	7	7
20	9	7	7	7
25	8	7	7	7
28, 32, 36	7	6	6	6
40, 50, 60	7	6	6	6

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TABLE A.6
ASTM A706 tensile strength requirements

a. Standard bar sizes

	Grade 60 [420]	Grade 80 [550]
Tensile strength, min, psi [MPa]	80 000 [550][A]	100 000 [690][A]
Yield strength, min, psi [MPa]	60 000 [420]	80 000 [550]
Yield strength, max, psi [MPa]	78 000 [540]	98 000 [675]
Elongation in 8 in. [200 mm], min, % bar designation nos.		
3, 4, 5, 6 [10, 13, 16, 19]	14	12
7, 8, 9, 10, 11 [22, 25, 29, 32, 36]	12	12
14, 18 [43, 57]	10	10

[A]Tensile strength shall not be less than 1.25 times the actual yield strength.

b. Alternate bar sizes

	Grade 60 [420]	Grade 80 [550]
Tensile strength, min, psi [MPa]	90 000 [620][A]	105 000 [725][A]
Yield strength, min, psi [MPa]	60 000 [420]	80 000 [550]
Yield strength, max, psi [MPa]	78 000 [540]	98 000 [675]
Elongation in 8 in. [200 mm], min, % bar designation nos.		
10, 12, 16, 20	14	12
25, 28, 32, 36	12	12
40, 50, 60	10	10

[A]Tensile strength shall not be less than 1.25 times the actual yield strength.

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TABLE A.7
ASTM A1035 tensile strength requirements

a. Standard bar sizes

Type	A1035 CL		A1035 CM		A1035 CS	
	Gr. 100	Gr. 120	Gr. 100	Gr. 120	Gr. 100	Gr. 120
Grade	**[690]**	**[830]**	**[690]**	**[830]**	**[690)**	**[830]**
Tensile strength, min, psi	130 000	150 000	120 000	120000	150 000	150 000
[MPa]	[900]	[1030]	[1030]	[1030]	[1030]	[1030]
Yield strength (0.2% offset),	100 000	120 000	100 000	120 000	100 000	120 000
min, psi [MPa]	[690]	[830]	[690]	[830]	[690]	[830]
Elongation in 8 in. (200 mm], min, %:						
bar designation No.						
3 through 11	7	7	7	7	7	7
[10 through 36]						
14, 18 [43, 57]	6	6	6	6	6	6

b. Alternate bar sizes

Type	A1035 CL		A1035 CM		A1035 CS	
	Gr. 100	Gr. 120	Gr. 100	Gr. 120	Gr. 100	Gr. 120
Grade	**[690]**	**[830]**	**[690]**	**[830]**	**[690)**	**[830]**
Tensile strength, min, psi	130 000	150 000	120 000	120000	150 000	150 000
[MPa]	[900]	[1030]	[1030]	[1030]	[1030]	[1030]
Yield strength (0.2% offset),	100 000	120 000	100 000	120 000	100 000	120 000
min, psi [MPa]	[690]	[830]	[690]	[830]	[690]	[830]
Elongation in 8 in. (200 mm], min, %:						
bar designation No.						
10 through 36	7	7	7	7	7	7
40 through 60	6	6	6	6	6	6

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TABLE A.8
ISO 6935-2:2015 tensile strength requirements

Ductility class	Steel grade	Specified characteristic value of upper yield strength R_{eH} MPa		Ductility properties		
				Specified characteristic value of R_m/R_{eH}	Specified characteristic value of elongation %	
		Minimum	Maximum	Minimum	A Minimum	A_{gt} Minimum
C	B300C-R	300	–	1.15	16	7
	B400C-R	400	–		14	
	B400CWR					
	B500C-R	500	–			
	B500CWR					
	B600C-R	600	–		10	
	B450CWR	450	$1.25 \times R_{eH}$ (min)		–	7.5
D	B300D-R	300	–	1.25	17	8
	B300DWR		$1.3 \times R_{eH}$(min.)			
	B350DWR	350				
	B400DWR	400				
	B420DWR	420			16	
	B500DWR	500			13	

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TABLE A.9
ASTM A615 bend requirements

a. Standard bar sizes

	Pin diameter for bend tests [A]			
Bar designation no.	**Grade 60 [420]**	**Grade 75 [520]**	**Grade 80 [550]**	**Grade 100 [690]**
3, 4, 5 [10, 13, 16]	3½ *d*	5*d*	5*d*	5*d*
6 [19]	5*d*	5*d*	5*d*	5*d*
7, 8 [22, 25]	5*d*	5*d*	5*d*	5*d*
9, 10, 11 [29, 32, 36]	7*d*	7*d*	7*d*	7*d*
14, 18 [43, 57] (90°)	9*d*	9*d*	9*d*	9*d*
20 [64] (90°)	10*d*	10*d*	10*d*	---

[A]Test bends 180° unless noted otherwise.

b. Alternate bar sizes

	Pin diameter for bend tests [A]			
Bar designation no.	**Pin diameter for bend tests [A]**	**Pin diameter for bend tests [A]**	**Pin diameter for bend tests [A]**	**Pin diameter for bend tests [A]**
10, 12, 16	3½ *d*	5*d*	5*d*	5*d*
20	5*d*	5*d*	5*d*	5*d*
25	5*d*	5*d*	5*d*	5*d*
28, 32, 36	7*d*	7*d*	7*d*	7*d*
40, 50, 60 (90°)	9*d*	9*d*	9*d*	9*d*

[A]Test bends 180° unless noted otherwise.

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TABLE A.10
ASTM A706 bend requirements

a. Standard bar sizes

	Pin diameter for 180° bend tests	
Bar designation no.	**Grade 60 [420]**	**Grade 80 [550]**
3, 4, 5, [10, 13, 16]	3*d*	3½*d*
6, 7, 8 [19, 22, 25]	4*d*	5*d*
9,10, 11 [29, 32, 36]	6*d*	7*d*
14, 18 [43, 57]	8*d*	9*d*

b. Alternate bar sizes

	Pin diameter for 180° bend tests	
Bar designation no.	**Grade 60 [420]**	**Grade 80 [550]**
10, 12, 16	3 *d*	3½ *d*
20, 25	4*d*	5*d*
28, 32, 36	6*d*	7*d*
40, 50, 60	8*d*	9*d*

TABLE A.11
ASTM A1035 bend requirements

a. Standard bar sizes

Bar designation no.	**Pin diameter for bend tests[A]**
3, 4, 5, [10, 13, 16]	3½*d*
6, 7, 8 [19, 225]	5*d*
9, 10, 11 [29, 32, 36]	9*d*
14, 18 [43, 57] (90°)	7*d*

[A]Test bends 180° unless otherwise agreed.

b. Alternate bar sizes

Bar designation no.	**Pin diameter for bend tests[A]**
10, 12, 16	3½*d*
20, 25	5*d*
28, 32, 36	7*d*
40, 50, 60 (90°)	9*d*

[A]Test bends 180° unless otherwise agreed.

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TABLE A.12
ISO 6935-2:2015 bend requirements

a. Bend test requirements

Nominal bar diameter d, mm	Mandrel diameter (max.)[a,b], mm
≤16	3d
16 < d ≤ 32	6d
32 < d ≤ 50	7d

b. Rebend test requirements.

Nominal bar diameter d	Mandrel diameter (max.)[a,b]
≤16	5d
16 < d ≤ 32	8d
32 < d ≤ 50	10d

[a] For nominal diameters larger than 50 mm, the mandrel diameter in bend tests shall beagreed between the manufacturer and purchaser.

[b] By agreement between the manufacturer and purchaser, larger mandrel diameters maybe used.

TABLE A.13
Modification factors for development of hooked bars in tension (ACI 318-19)

Modification factor	Condition	Value of factor
Lightweight, λ	Lightweight concrete	0.75
	Normal-weight concrete	1.0
Coating, ψ_e	Epoxy-coated or zinc and epoxy dual-coated reinforcement	1.2
	Uncoated or zinc-coated (galvanized) reinforcement	1.0
Confining reinforcement, ψ_r	For no. 11 and smaller bars with $A_{tr} \geq 0.4A_{bs}$ or $s \geq 6d_b$	1.0
	Other	1.6
Location, ψ_o	For no. 11 and smaller diameter hooked bars: 1. Terminating inside column core with side cover normal to plane of hook ≥ 2.5 in., or 2. With side cover normal to plane of hook ≥ $6d_b$	1.0
	Other	1.25
Concrete strength, ψ_c	For f'_c < 6000 psi	f'_c /15,000 + 0.6
	For $f'_c \geq$ 6000 psi	1.0

Note: a. s is minimum center-to-center spacing of hooked bars.
b. d_b is the nominal diameter of hooked bar.

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TABLE A.14
Modification factors for development of Headed bars in tension (ACI 318-19)

Modification factor	Condition	Value of factor
Coating, ψ_e	Epoxy-coated or zinc and epoxy dual-coated reinforcement	1.2
	Uncoated or zinc-coated (galvanized) reinforcement	1.0
Parallel tie reinforcement, ψ_p	For no. 11 and smaller bars with $A_{tt} \geq 0.3A_{hs}$ or $s \geq 6d_b$	1.0
	Other	1.6
Location, ψ_o	For headed bars: 1. Terminating inside column core with side cover to bar ≥ 2.5 in., or 2. With side cover to bar ≥ $6d_b$	1.0
	Other	1.25
Concrete strength, ψ_c	For $f_c' < 6000$ psi	$f_c'/15{,}000 + 0.6$
	For $f_c' \geq 6000$ psi	1.0

Note: a. s is minimum center-to-center spacing of hooked bars.
b. d_b is the nominal diameter of hooked bar.

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